TEXAS Highways

Short Course Issue

November, 1954

Accounting

Introduction . . .

This volume of approximately 200 pages is the fourth and largest of our Short Course issues. Papers and discussions presented by highway engineers, maintenance and administrative personnel, A & M faculty members, and leading businessmen are reproduced herein for the benefit of personnel unable to attend the course. Not all of the informative discussions of the twenty-eighth Short Course are included since some speakers used notes rather than prepared texts, and it was also impossible to include any transcripts of the many floor discussions.

This year marked an inovation in the format of the course, with the first day taken up with a General Session at which topics of broad interest were covered. On the following days various group discussions on the many phases of highway design, construction, maintenance, and finance were held for those interested in attending them.

The purpose of the new program was to determine whether smaller individual sessions would be more valuable as instructional media than large formal meetings. Consideration is being given to planning future programs in this way.

Our cover this month typifies the interdependence of all divisions and districts of the Highway Department, and sums up the purpose of the annual Short Course. This mural was painted by Willard Edison Pressnalle in the office of Knox E. Dixon, Maintenance Foreman in Anahuac County. The motto Ave Stella Gloriosa refers to the Lone Star State.

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TEXAS Highways

The purpose of this publication, "Texas Highways," is to furnish a medium for presenting practical ideas and information. Employees are invited and urged to submit suggestions and relate experiences which would benefit or be of interest to Highway employees. These conclusions and data are not necessarily endorsed by the Highway Department nor are they to be construed as instructions.

Photographic services are available and other assistance may be obtained, if desired, in preparing material for submission. All material and comments should be directed to the Information and Statistics Division, Austin 14, Texas.

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COMPILED AND EDITED
Information and Statistics Division

Twenty-eighth

Annual Short Course

TUESDAY, SEPTEMBER 7

Morning: Ballroom Memorial Student Center

8:00 Registration, Memorial Student Center

Chairman: Dr. S. R. Wright, Head
Civil Engineering Department
A & M College of Texas
College Station, Texas

10:30 Invocation: Rev. Norman Anderson, Pastor
A & M Presbyterian Church
College Station, Texas

Address of Welcome
Dr. David H. Morgan, President
A & M College of Texas

Response
Tyree L. Bell, President
Austin Road Company
Dallas, Texas

11:00 Texas Highway Needs
E. H. Thornton Jr., Chairman
Texas Highway Commission
Galveston, Texas

11:30 A Look Ahead
Pyke Johnson, President
Automotive Safety Foundation
Washington, D. C.

Afternoon: Ballroom Memorial Student Center

Chairman: Frank W. Cawthon, District Engineer
Texas Highway Department
Dallas, Texas

1:45 The Nation's Highway Future
D. C. Greer, State Highway Engineer
Texas Highway Department
Austin, Texas

2:15 Highway Construction Future
E. B. Cape, President
Gulf Bitulithic Company
Houston, Texas

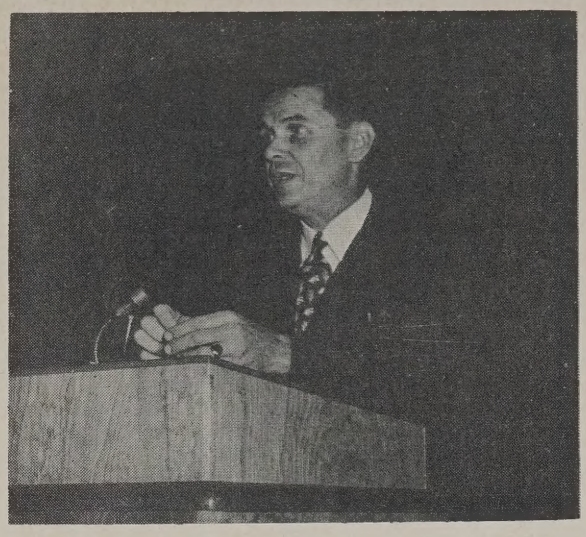
Intermission

3:00 Highway Loadings
Henson K. Stephenson, Research Engineer
A & M College of Texas

3:30 The Malad Test Road
Frank H. Scrivner, Supervising Research Engineer
Texas Highway Department
Austin, Texas

4:00 Future of Highway Short Course
Fred J. Benson, Professor
Civil Engineering Department
A & M College of Texas

7:00 Annual Banquet, Sbisa Hall
Toastmaster: D. C. Greer
Texas Highway History



Address of Welcome --
Dr. David H. Morgan, President
A & M College of Texas

Response --
Tyree L. Bell, President
Austin Road Company, Dallas

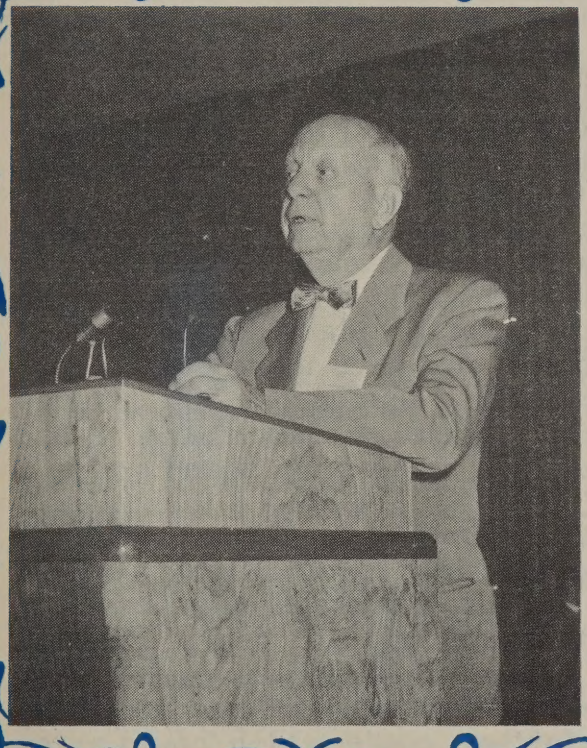
Dr. David H. Morgan, President of A & M College, welcomes D.C. Greer, State Highway Engineer, at the beginning of the Twenty-eighth Annual Highway Short Course.

Chairman --
Frank Cawthon, District Engineer
Texas Highway Department

The Nation's Highway Future --
D.C. Greer, State Highway Engineer
Texas Highway Department

A LOOK AHEAD

Pyke Johnson, President
Automotive Safety Foundation
Washington, D.C.



In the dramatic onsweep of the machine age which has marked life in the United States since World War II, Texas has played a spectacular part.

Where in 1939 farm labor was 30 per cent of all Texas employment, today it is 16. In the interval, notably from 1945 on, tractors on the farms have doubled in number. Today half of all farms in the state are mechanized. Output per acre and per man-hour has been rising steadily.

Total income has risen 348 and per capita wealth 251 per cent since 1940. The number of men employed in manufacturing industries has more than doubled, yet through the constant advance in machine methods, the value added by manufacture has multiplied more than five times.

More recently, from 1950 through June 30, 1953, the investment in expansion of defense facilities has reached about $2 billion.

Too, there has been a phenomonal growth in motor vehicle use, another important index of the progress of the machine age. Both cars and trucks now in use are double the number in 1940, a figure that sharply exceeds that of the United States as a whole for passenger cars and is about the same as the national average for the truck. Where the total vehicle mileage was 13.5 billion in 1940 in Texas and 16.4 billion in 1946, by the end of last year, the total was practically double that of 1946.

When travel figures were compiled for the Texas Highway Needs pamphlet, an impressive total of 86 million miles was being driven every day in Texas. More recent national estimates, show that even that incredible number has been surpassed.

Nor is the end in sight. Recent government reports forecast 100 million motor vehicles in the United States by 1975. Will Texas have its share? I leave it to you to answer that one.

On all sides and in almost every activity, Texas is moving forward at a prodigious rate. As industrial chemistry has progressed, the state's

vast natural resources are being developed at a constantly accelerating pace.

The impact on the way of living of the Texan has been profound. With one vehicle to every two residents, the radius of urban influence has been greatly expanded. Cities have literally exploded their earlier boundaries. Schools have been consolidated. Farmers have been brought close to market. The line between rural and city life has become faint indeed.

President Eisenhower's message to the governors on July 12 is peculiarly appropriate to the Lone Star State. He said:

"We live in a dramatic age of technical revolution through atomic power and we should recognize the fact that the pace is far faster than the simpler revolutions of the past. It was a very long generation from the Watts steam engine to a practical steam locomotive. It was less than nine years from the atomic bomb to the launching of an atomic-powered submarine. We have seen a revolutionary increase in opportunity, comfort, leisure, and productivity of the individual.

"Thirty years ago, the machine economy was almost entirely limited to factories and transportation. Today it is in every area of living, even in the garden patches and the front lawn." And he might have added for our northern states, and on our snow-covered sidewalks!

All of that is on the credit side and much more could be said in glowing terms about that part of the world in which we as Americans are privileged to live.

Let's try to put it all into more personal terms and to consider the reverse side of the coin as well.

Suppose, for example, as a blueprint for action in what follows in this look ahead, each of you present paused for a moment to ask yourself these two questions:

"What would happen to me, to my family, to my business, if suddenly we were to be deprived of the use of our car, the truck, and the bus?"

"In the light of our constantly increasing use of these vehicles, can I, my family, and my business, possibly afford to be without adequate facilities for their safe and efficient use and storage -- storage in this case being parking?"

The answer to the first question comes quickly. It is also one way of drawing a line between life as it is today in the United States and almost all of the rest of the world.

The answer to the second question is equally clear, but it forces all of us to face up to present-day realities. It poses one of the stiffest civilian, social, financial, engineering, and educational problems which still lie ahead of us.

Put bluntly, the position we are in today in Texas, Maine, Oregon, Ohio, Florida, every other state is this:

Neither in facilities nor in skills of use have we yet been able to match the technological and selling genius which has placed 56 million motor vehicles on our roads and streets.

In fact, not sensing the change which is taking place in the lives of all of us, we have made day-to-day, piecemeal approaches to issues which require research and broad policies as a basis for sound action.

The result has been a terrible toll in life, limb, and property on the public highway, together with congestion and its resultant huge costs to industry and private users growing out of the deficiencies created by inadequate transportation facilities. The further costs which could arise in case of atomic warfare are almost beyond human imagination in the horrors they conjure up of millions trapped in heavily populated areas.

Let me be specific. Because our city planners, councils, architects, and realty owners did not sense the need for off-street loading and unloading facilities for trucks, staggering sums have been invested in buildings which now must be modernized or take their place in blighted areas.

Because we have not anticipated the need for taking care of the vehicle at rest, we have used costly street facilities for storage space, allowing a single vehicle at times and places to eliminate a full lane and obstruct a second lane, thereby retarding the

movement of hundreds of vehicles per hour.

Because we did not foresee that practically all home owners and apartment users would possess vehicles, we have not provided full residential off-street parking facilities nor have we zoned districts with a view to making life in new subdivisions more pleasant for the occupants and safer for their children, and one might think, more profitable for the owner.

Further, we have superimposed modern traffic on horse and buggy street layouts and so must face squarely drastic street modernization at heavy cost to save the circulatory system of our metropolitan areas, their realty values and position as business center.

Where we have built new highways or bypasses, we usually have failed to provide adequate funds to acquire needed rights of way. So we find all kinds of business structures and billboards encroaching upon the right of way. The capacity of the road for safe movement is diminished.

One is tempted in viewing situations like these to ask ironically: "Do we know even yet what we are building roads for? Is it for storage, for service to abutting property or for movement? Isn't it about time we decided?

During the great depression of the 30's, we diverted huge sums to leaf raking or its equivalents. In World War II, all road building except that immediately essential to the war, was stopped by order of the President. The records of roads constructed and in place -- the life line of highway progress -- show that we did not fully regain the momentum so lost until ten years later. Meanwhile people were buying faster and heavier vehicles by the millions. Inflation had set in.

It is a statement of fact that in the postwar era, the manufacturers have built more miles of vehicles bumper to bumper than the highway engineers have been able to build of roads. True, roads are built in multiple lanes. True, also, vehicles don't move bumper to bumper safely nor are the roads always built where the mass traffic is.

In consequence, all over the United States we are suffering from critical deficiencies in our roads which will require more billions of dollars annually to their rehabilitation, than all we are now spending.

"We require," says the President of the United States, "a grand plan for a properly articulated system that solves the problems of speedy, safe, transcontinental travel -- intercity communication -- access highways -- and farm-to-market movement -- metropolitan area congestion -- bottlenecks -- and parking."

"The penalties of our present appalling inadequacies," he says, "warrant the expenditure of billions to correct them," and so he advocates a new program of $50 billion in ten years over and above existing expenditures as a "goal toward which we can -- and we should -- work."

That statement made, he then calls upon the governors of the states to join in studying this problem and in recommending to him "cooperative action so that he, the President, can submit positive proposals to the next session of the Congress."

It is a sufficient earnest of the President's view of the seriousness of the issue to take note of the fact that his message came only a few weeks after the passage by Congress of the largest Federal road bill in our history -- a bill which increased existing authorizations of $575 million annually to nearly $1 billion. In passing, this law puts heavy responsibilities on each state to step up to its share of the financial and administrative problems involved in this already enlarged effort.

It is a sufficient indication of the attitude of the governors to note that immediately, they called upon the American Association of State Highway Officials to meet with them to consider this problem; that such meetings have already been held; and that gradually a "Grand Plan" is being evolved which will be ready by the next Congress. Meanwhile the Bureau of Public Roads, in cooperation with the states and the Council of State Governments, is proceeding with

broad studies of the facts underlying the needs as a basis for final policy action.

It is evidence of the universal demand for improved roads that generally the public reaction as voiced by the press, radio commentators, magazine publishers, other media, and interested groups, has been strongly in support of the imaginative approach of the President's "Grand Plan."

The real test will come, however, as the effort is made to give legislative and administrative substance to his outline.

Then we may expect that while all of us as individuals will recognize the imperative needs which lie behind his proposals, as groups we will probably react as we always have in the past. We will want the other fellow to pay the costs, or most of them, while insisting that the road shall go where we want it.

That approach cannot possibly be successful in a project the size and cost of this one. Individually and collectively, we must raise our sights. We must recognize that we cannot have roads unless we are ready to pay for them one way or another. We can only demand that they shall be built and held to necessary traffic requirements and that those requirements shall be determined by detailed study of all the facts, not on political preferment.

Parenthetically, three major errors in semantics have hung over the highway program for years. Let's put them on the table.

1. The use of the term "Aid" as if Federal funds for roads were a subsidy instead of a recognition of a constitutional obligation to pay a fair share of the costs due to Federal uses of the roads. Actually, if any level of government has been subsidized in the road program, it is the Federal, particularly, in the light of today's vast military transportation problems.

2. The idea that we have "free" roads is an age-old fallacy. Actually, every filling station is a toll collector and so is every other collector of any special motor vehicle tax.

3. Because government collects the charge for roads, we call it a tax and frequently set ourselves up in violent opposition. Anyone who buys a railroad ticket knows it includes costs of right of way, maintenance, equipment, operation, and a profit to the company. Basically, there is no difference between that charge and the road tax. Both are service charges. Both must be met.

The real problem is how to get needed roads quickly, at the lowest cost. Properly located, designed, constructed, and maintained, we all know that it will cost us less in the total family budget to have them than to go without them. It may even save our lives. Those are the central facts.

The rest is a matter of cutting the elephant into pieces, seeing how we can allocate costs most equitably as between capital expenditure and current costs; as between different levels of government; and as between different classes of beneficiaries. In this latter group, note should be taken of the amazing awards which realty has garnered from such developments as the Gulf Freeway.

At the Federal level, basic administrative policies are clean cut and a more than thirty-year highway program of cooperation between the Federal and State governments attests its success. States rights have been maintained. Funds have been directed to roads of major public concern. There has been no dispersion or diversion of authorized funds. The work has been done almost universally by private contract. A great system of highways has been built without scandal; rendered obsolete only by the constantly advancing technology of the manufacturers, wars, inflation, and an ever-broadening use without collateral development of the roads.

A practically unanimous, nonpartisan vote in both houses of Congress is evidence of public confidence in the job that has been done and faith in the principles contained in the Federal Aid Highway Act.

Nationally, the largest questions grow out of the relatively small Federal part in the financial phases of the highway program.

Thus far the Federal payments have

ranged around 12 per cent of the total. Actually, as today's law is written, the annual payment of $175 million toward the interstate system of 40,000 miles would require for the completion of those roads -- the most important in the nation -- anywhere from 50 to 75 years.

We can't and won't wait that long. So we see states having a heavy interstate traffic taking the position that if these roads which are primarily a national responsibility are to be built without delay, then they must be built as toll roads.

It follows that one of the large issues ahead is whether users are to assume the entire burdens of such roads when they could be built just as quickly by state highway departments and at lower costs to the user, especially if the Federal government were to put the weight of its borrowing power behind the construction of such "free" interstate roads.

Collateral issues are many. One is the view strongly expressed by municipal authorities that the Federal government has not paid its share of those major arterial urban roads which are an integral part of the interstate system. Along with that goes a point of equal importance. Even if feasible plans could be developed for urban toll roads, they say the more desirable and less expensive course is "free" roads. That means, inevitably, long-term financing. Houston has one solution in its proposed provision of expensive rights of way from long-term bond issues as the city's share.

Another issue is the view that interstate commerce should be held free from any undue restraints and consequently, that states should not be permitted to assess against interstate travel, rates which will bring huge profits. It is pointed out that such practices would not be permitted in the case of railroad and other interstate movements.

A third point of pressing importance is the creation of dual government agencies with all of their implications of government waste and undue costs. Here is an issue of vital moment to the entire highway program. Already we have seen separate turnpike authorities created whose function, once the road is built, is simply to maintain it in profitable operation. Free from the usual legal restrictions that hamper state highway departments in getting engineers, they have raided them at a time when highway departments at all levels are having a hard time maintaining imperatively needed personnel. In one department, for an example of the difficulty to get new men, out of a total of 200 engineers, less than ten are under 35 years of age.

If the engineering job is to be done economically, efficiently, and intelligently as it has been in the past, the present is no time to duplicate functions. On the contrary, the challenge that faces the universities, Congress, and the legislatures is to get highly trained young men into the highway service. Compensation and retirement funds must be increased or as taxpayers, we will have to pay the costs in inadequate work.

The President's pronouncement is that the program must be stepped up. The governors agree. It is a requirement of paramount importance that everything possible shall be done to enlist not alone engineering but all the other skills of a job which bear so importantly on our way of living as this one.

Otherwise, the highway engineers of the country could well wake up one day to charges of inefficiency, waste, and neglect in the performance of their duty.

Of all the issues before you of the university and engineering fields, probably none is of as far-reaching consequence as this human one -- the right man in the right place at the right time.

Along with this practical consideration goes another. Our highway laws, like other policies, frequently have been amended on a piecemeal basis. Their language often is not only confusing and overlapping but actually places obstacles in the way of getting the job done despite the clear intent of the people and their legislators.

A single illustration of vital consequence is the right-of-way

laws. Provision must be made to enable both states and local authorities to obtain adequate rights of way for important highways for several years in advance of construction. Further, the right to acquire first and to adjudicate costs later is a "must" if the purpose is to get the project underway without delay.

The position varies sharply with the state. In some, as of today, permission is not granted even for controlled access yet this is a necessary part of the acquisition of the right of way itself.

The Federal government has a direct interest in getting those roads of national importance built to needed standards. The right to use Federal funds for such acquisitions and the insistence on controlled access where needed and feasible as a prerequisite to the use of Federal funds would free the hands of engineers to get on with the job.

As we look ahead in the light of what has been achieved in the past, these points appear in strong relief:

1. The first demand on the public is for safe and efficient transportation of all kinds. If goods, for example, can be moved more rapidly and less expensively by "piggy-back" service over our railroads, then by all means that service should be developed. If city transit services can attract back private motor-car users, they should be free to do so. But in all events, competition should be maintained. The determining factor at all times must be the desires and needs of the customers; no punitive action in favor of any purveying group.

2. The public is not concerned with engineering terminology in road construction and use. As motorists you and I are only interested in getting from Point A to B safely and comfortably. Those points today may be from one farm to another; from home to shopping point; from one end of the country to the other, or even beyond. We don't know and don't care whether the road we travel is in the primary, secondary, or tertiary classification.

3. We want the road. We want it to be built at the lowest cost and well maintained. We don't like to pay taxes but we will if we can be assured that you, the legislators, the road builders, are spending our special taxes to help us travel more safely and more economically.

Back of this call for all kinds of roads is the insistent need for the close integration by law of the thousands of independent road jurisdictions which have existed in our country since the early days. We need as well the same close cooperation between the universities and state highway departments as is evident here at Texas A and M. We need it because road building is a dynamic job and we must have research as well as trained men on the job.

The close cooperation which has existed between Federal and State officials under the Federal Act must be extended throughout the field. Laws must be modernized. Every obstacle which stands in the way of getting the job done must be eliminated. Our system of highway financing must be revised to meet today's needs. There must be public awareness not so much of the needs as of what is being done to meet them. The problem of highway safety should be meshed in much more closely to the engineer's job than it has been in the past because safety remains one of the principal aims of an adequate highway system. Educators, research men, enforcement agencies, registration officials, and the drivers, are all part of the squad.

As we face up to the great new goal set down by the President, the first step should be one of elimination of all of the blocks which may stand in the way of getting the job done.

There is a place in this magnificent undertaking for all of us, because it can only be realized as it becomes an ALL-AMERICAN EFFORT.

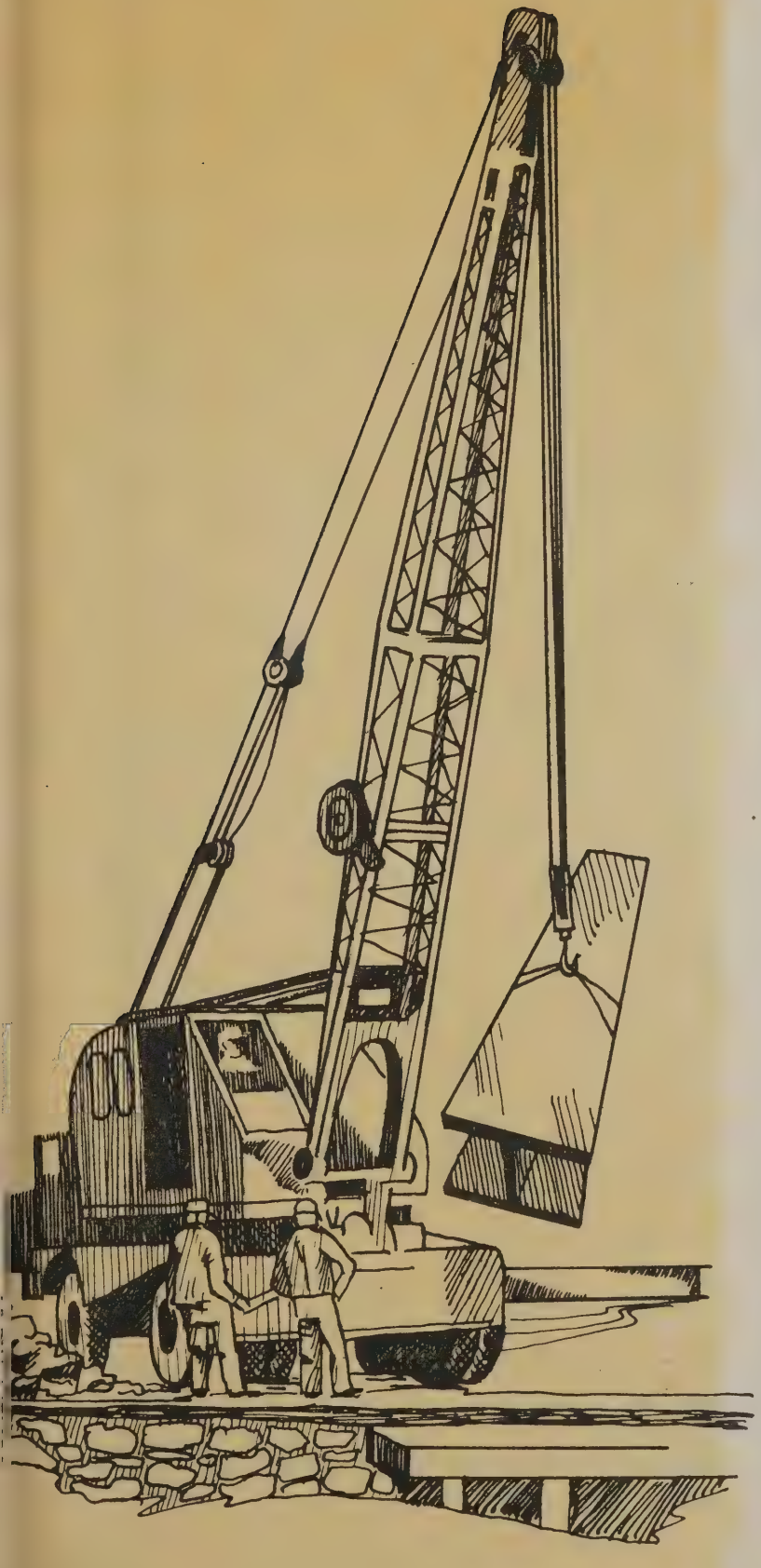
You who are legislators, educators, or engineers and have the legal and moral responsibilities to carry on, are our first team.

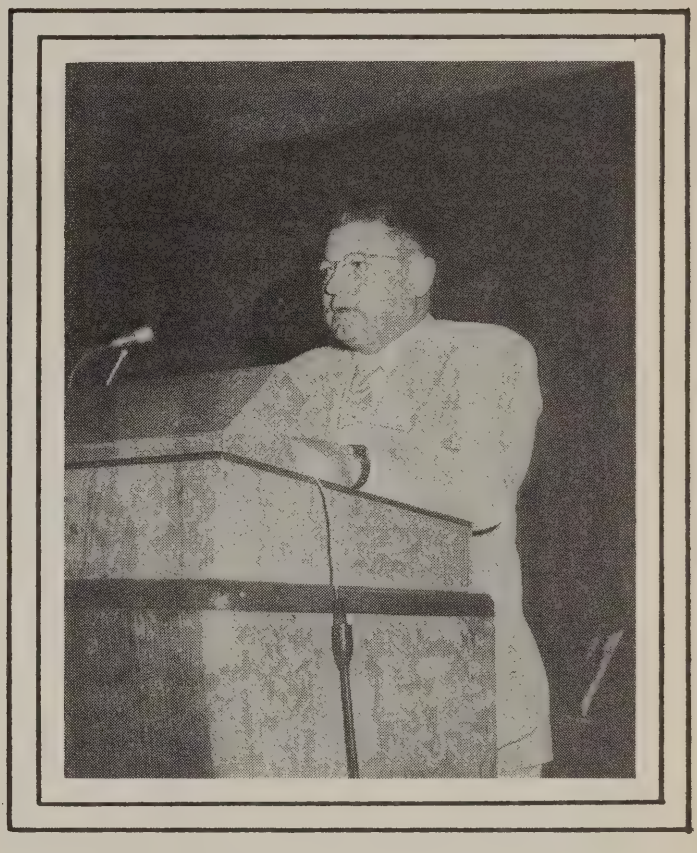
It is up to the rest of us to give you our full backing.

That you are assured as you go forward.

As the President has said, this job is essential, but it can only be done in the "closest cooperation."

HIGHWAY CONSTRUCTION FUTURE





E. B. Cape, President
Gulf Bitulithic Company
Houston, Texas

It is a pleasure to attend this the Twenty-eighth Annual Highway Short Course, and I would like to take this opportunity to thank the A & M College and the Texas Highway Department for making this meeting possible. It is always a great help and an inspiration to be able to attend and be a part of these fine programs.

Any one of the very able speakers who have preceded me could give a world of information as to the need for increasing highway construction. Mr. Thornton, Mr. Johnson, or Mr. Greer could easily use the entire day in discussing this very timely subject.

I would like to personally thank Mr. Thornton and the other members of the Highway Commission, on behalf of the construction industry, for their untiring effort in presenting our highway problem to the people of Texas.

In March of this year Mr. Greer and the Commission announced that the current fiscal year marks the beginning of the second billion dollars of highway construction to be performed by Texas contractors under the State Highway Department. We are proud of our record and of our association with the Texas Highway Department in this work.

This first billion contracted plus force account construction and other assets places the investment of the Texas Highway System at about one and one-half billion dollars. This is a lot of money and we have a lot of highways, but latest figures published by the American Association of State Highway Officials show that Texas has about 30 thousand miles of main highways that need improvements at an estimated cost of nearly one and one-half billion dollars. We need to spend today on our highway system an amount almost equal to the amount spent by the Highway Department in the 37 years from its creation in 1917 to the present. We are told that this lost ground has resulted in an economical loss of 150 million dollars a year as a result of highway accidents alone. Add to this amount lost time, wasted fuel, etc. and one can easily see that the old saying which we have been told so many times is a true statement of fact. I would like to repeat this statement again, "We pay for highways whether we have them or not."

The approximate 50 per cent increase in the 1954 Federal Aid Highway Act will be a help and President Eisenhower's proposed 50-billion dollar, 10-year program, in addition to regular Federal Aid, will be a big boost should it become law in 1955, but the fact remains that we in Texas must provide our share of matching dollars as well as provide for the ever-increasing maintenance load.

I believe the people of Texas will provide the necessary funds, at the State level, to insure the necessary funds to build and maintain a modern and adequate State Highway System. To substantiate this belief I can cite (1) The toll road authority, (2) County bond elections, and (3) City bond elections. Each of these, along with the increased Federal Aid, will be a help but the toll road cannot replace public facilities, the county bond issue does not relieve the cry for increased farm-to-market roads, and the large city bond issues provide scarcely more than sufficient funds to acquire right-of-way and therefore cannot eliminate the need for vast urban expansion.

Now let's look for a minute at the construction industry and the future. The construction industry has been tried and found not wanting. I cite first the great construction program which was undertaken during World War II. In spite of tremendous hardships of reduced personnel, material shortages, equipment and parts shortages, we were able to complete the necessary facilities in what we think was a record time.

Since the creation of the State Highway Department the construction industry has kept pace with an increasing highway program each year. To illustrate this growth remember that in 1935 the State Highway Department started less than 21 million dollars worth of work as compared with over 132 million which will be started in 1954.

We believe the personnel situation is now stronger in the construction industry than it has been at any time in the past. We feel that most contractors have old and tried employees who will be able to cadre new outfits. We believe the supply of operators will be able to hold pace and that the number of young graduate engineers entering the construction industry will be of help in absorbing an increased working load.

The speaker is neither an equipment expert nor a prophet, but for an insight into the role equipment will play in the construction future one has only to look at the past. Take, for example, the tractor. Caterpillar has this year celebrated the fiftieth

anniversary of the crawler tractor. This means that the entire development of this type equipment has been within the lifetime of a great number of those that are present here today. It has been a very few years for any old-timer since the "Best 60" was the zenith in power. In these few years we have seen the old "60" replaced with bigger and better machines and in the years to come we feel sure the TD 24s and the D9s will likewise be replaced. Within these few years we have seen the crawler tractor adapted to bulldozers, root rakes, scrapers, end loaders, elevating loaders, huge compactors, and many other uses. We have seen a like advancement in practically all of our equipment. We have seen arc welding, Gyro-Flo compressors, diesel engines, portable electric power, mobile batching plants, transit mixers, twin-power scrapers, rubber-tired cranes, and many other advances come into prominent use. One of the most promising things to come to our mind is the concrete paver that will lay its own forms as it moves along the subgrade. I cannot speak for the construction industry as a whole, but our company has added a large amount of equipment in the last few years. For example, we have replaced two hot mix plants and added a third. We know other contractors have expanded in this as well as other types of equipment. We believe and have confidence that this parade of progress will be equal to any accelerated program of work offered to the construction industry.

I have talked briefly about personnel and equipment and feel that the suppliers for the construction industry have likewise adequately kept pace with the accelerated program the last 37 years and that they again will be able to meet the demands of the future.

When the subject of an expanded highway program is discussed some express the fear that the law of supply and demand might increase prices, but just the opposite is more than likely to happen.

As the amount of work increases, everyone keeps bidding. All of us are guilty of sometimes bidding more work than we can handle and when this happens we buy additional equipment or subcontract part of our work. Our subcontractors take on additional work and they in turn buy additional equipment and thus the cycle is started. Soon we find subcontractors who are bidding work and in desperation are bidding at cost or below cost to hold an organization and make equipment payments. The thing they forget is that the equipment they are wearing out to make payments should in reality be providing for the orderly process of replacing equipment. As a matter of fact I am of the opinion that the construction industry is at this time overexpanded and that unless additional work is made available there may be some hard times ahead. A few reasons for this belief are: (1) Cost of work has been going down for several years while the cost of labor and materials has been going up; (2) There are still just as many bidders, or perhaps more bidders, per job than there were two years ago; (3) Some of the large organizations such as the American Road Builders' Association have working committees to study the possibility of maintenance by contract in order to alleviate overexpansion by contractors.

Hence this word of caution, a large increase in highway construction could result in more broke contractors than rich ones.

We hope when the day comes that additional funds on the State level are made available these funds will be adequately protected by constitutional amendment. I am not going to deal at great length on the subject of legislative needs for our industry but I would like to mention a few thoughts that could have a great effect upon all of us present here today. Some of these thoughts are as follows: (1) Protect open shop highway contractors; (2) Protect engineers from the union; (3)Realistic wage scales; (4) Insurance costs; (5) Taxes. Any of the above listed items could materially affect the cost of our doing work. For example, should we be forced into wage scales of the magnitude of the building trade for highway contracts, then we will

have lost tremendous ground in our construction program, notwithstanding any additional funds that may have been made available.

I think the present method of taking bids is sound and have only one suggestion to make. I do not think this is the time or place for any full-scale discussion, but I would like to present a point to see if it will stimulate future thought and perhaps discussion at a later date. My thought is that the state should institute a requirement that all bids must be submitted by mail and bids to be opened must be postmarked at least the day before opening date. If time did not permit the opening and reading of all bids on the first day then the opening and reading would be continued on the second day until all bids are read. This perhaps may not be in accordance with present law and it may never happen, but my reasons are: (1) There is a tremendous pressure on all bidders on the night before a letting. Many material prices are not stabilized until the wee hours of the morning and many times it is necessary to work all night in order to properly prepare a bid; (2) It is necessary to virtually establish a temporary office in Austin for each letting. We enjoy coming to Austin but feel that any bidder can do more competent work in his own office with his own office personnel and office machines; (3) We love the peddlers when we need them, but there is a tremendous gathering in Austin for each letting and a lot of people actually come to sell, entertain, or be entertained and not to bid work; (4) Some bidders in the heat of excitement are inclined to bid the second day against some other contractor's prices of the preceding day although job conditions may be entirely different. This eratic bidding usually results in a nonprofitable venture for the contractor and a lot of grief for the engineer.

Any time we get a group of engineers and contractors together the subject of engineer-contractor relations is apt to come up. A great number of us obtained our formal education in these same buildings here at A & M College, and from some of the same personnel who are appearing on this program. A number of you have worked for contractors and a number of us have worked for the State Highway Department. I think it one of the greatest fallacies of our time to feel that contractors are the natural enemy of the engineers and vice-versa. Actually I believe that among the supervisory personnel in our company we have a higher percentage of college-trained engineers than exists in the average highway residency. I feel that the relation between the contractor engineer and state engineer should be that of a partership and when this relationship exists the best possible job can be done. There has been some excellent progress made along this line by the A. A. S. H. O. in its joint committee work with the A. G. C. On the other hand an individual with a negative personality can hurt a job. This is true whether he is a contractor engineer or a state engineer. Therefore, I can only suggest that both contractors and the state concentrate on this phase of the problem and forget the old worn out subject, contractor-engineer relation.

I am sure that you have noted a spirit of optimism in this paper when discussing the ability of the construction industry to handle any expanded highway construction program. I am confident the industry can and will handle any work of the future which may be offered for bids. Notice that I did not say which contractor could or would handle any expanded work because bidding today is highly competitive. We in the industry would not have it otherwise and there is one thing you can always be assured of. Any reduction in cost, whether due to vicious bidding, new and better equipment, contractors know-how, or some other source, will always be passed right back into the highway fund. You can't survive in this business unless you bid them hard. I have tried to mention a few points which show that I believe the construction industry can adequately handle your work in the future insofar as personnel, equipment, supply, etc. are concerned. The question is can you engineers offer the same?

HIGHWAY LOADINGS

Henson K. Stephenson, Research Engineer
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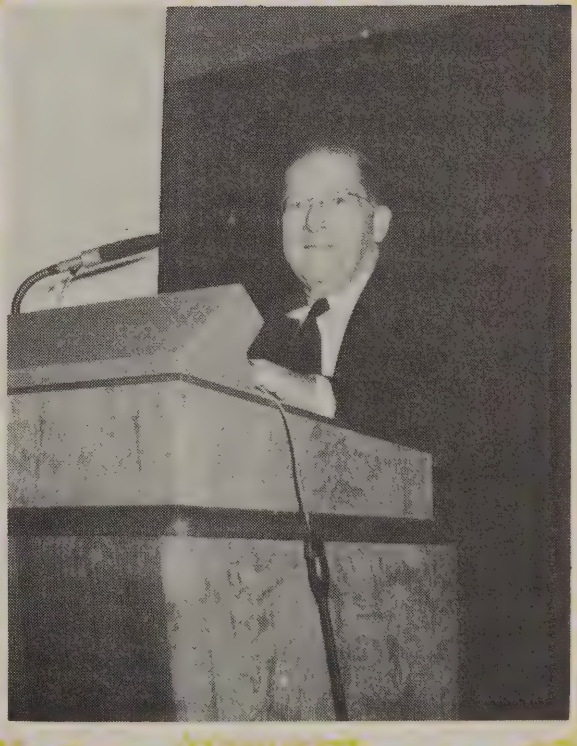

Historical Background: In the field of civil engineering it is sometimes difficult to realize how very recent are the developments that took the design of structures out of the hands of craftsmen and put it into the hands of scientists and engineers. Without going into detail a little historical perspective can be established by pointing out that the first attempt to investigate the strength of any structural member mathematically was not undertaken for more than a hundred years after the discovery of America in 1492 by Columbus. In fact it was not until after the settlement of Jamestown (1607), at about the time that the pilgrims landed at Plymouth Rock in 1620, that Galileo (1564-1642) undertook to investigate mathematically the strength of a rectangular cantilever beam with a single concentrated load at its end. Although the formula he derived was not correct, his investigation, which became known as "Galileo's Problem," serves to mark the beginning of present-day theory of structures based on the stress-strain relations in elastic

solids.

For the next 200 years Galileo's problem remained unsolved until the first correct analysis was given by C. L. M. Navier (1785-1836) in a paper presented to the French Academy in 1820 which from an historical standpoint is but a short time ago. Although the problem was not solved during this 200 years, the interval between Galileo and Navier was very important because it was during this time that the fundamental principles and methods, upon which modern science has been founded, were discovered and developed. Among the more important events of this period were the development of the science of the calculus and formulations of the laws of motion by Sir Isaac Newton (1642-1727), which were published in his book "Principia" in 1687; the discovery in 1660 and publication in 1676 of the law – now known as Hooke's law – by Robert Hooke (1635-1702), which established the relation between stresses and strains in a material, and the loads or forces required to produce them; and the introduction of the concept by Thomas Young (1773-1829) in 1807, that each material had a characteristic modulus of elasticity.

In the matter of compression members, it will also come as a surprise to many to learn that it was not until 1744 – just a short time prior to our own American Revolution – that the first mathematical analysis of column action was developed and published by Leonhard Euler (1707-1783). Euler solved the problem of the load-carrying capacity of an ideal column made of a homogeneous material of uniform section with position-fixed but unrestrained ends whose load is directed along the centroidal axis. Euler's solution, however, does not take into account any initial crookedness or other imperfections, and makes no allowance for unavoidable eccentricity of loading. And though the Euler formula indicates the ultimate load-carrying capacity of an ideal column under ideal conditions, it does not provide the means for analyzing the actual stresses produced in practical members. Then it was over 100 years before the next contribution to the mathematical analysis of columns was developed by H. Scheffler and published in 1858. In his development of the Secant formula, he gave for the first time the complete solution for an eccentrically loaded column, with equal eccentricities on the same side of its axis. Then after almost another 100 years the general solution for eccentrically loaded hinged end members was developed by D. H. Young and published in the 1936 ASCE Transactions – and that was less than 20 years ago. Even today much remains to be done because as yet no one has published a satisfactory mathematical procedure for the stress analysis and design of fixed or restrained ended columns.

It will definitely come as a surprise to many to learn that the first correct analysis of the stresses in a truss was developed but little over 100 years ago by an American named Squire Whipple and published in his book, "Work on Bridge Building" in 1847. Whipple is also credited with having built the first all-metal railway bridge in the United States, which was completed in 1853. This bridge was constructed using cast-iron compression members and wrought-iron tension members because structural steel was unknown at that time. The Bessemer process for making steel was invented in 1855 and the open-hearth process was invented a little later in 1861. However, the development of the steel industry was slow and wrought iron and cast iron remained the almost universal bridge metals until 1880; but between then and 1890 open-hearth steel came into vogue, supplanting wrought and cast

iron almost entirely.

The first railroads in the United States were built in the 1830's. They used horses for motive power and it required some time to demonstrate the steam locomotive to be more satisfactory and efficient. Once the steam locomotive was established and rail lines became longer, the demand for an abundant, strong, and inexpensive metal for rails and bridges became more pressing. The development of the steel industry was in response to these needs. The railroads opened up the Middle West and the Far West and by the late 1890's and early 1900's had penetrated the remotest parts of the nation. Along with these developments had come the telegraph, electric light, telephone and bicycle. The internal combustion engine had been invented and the first automobiles appeared in the late 1890's. The original "good roads" movement was started in the 1870's and, contrary to present-day opinion, it was inspired by the bicycle rather than the automobile: the original "good roads" movement preceded the automobile by perhaps 20 years.

Early Highway Loadings: And what, one may ask, do the preceding historical developments have to do with highway loadings? The answer is that they not only serve to show the progressive combinations of events which provided the conditions necessary to the generation of highway loadings but also to emphasize the fact that modern science and engineering are of very recent origin.

The opening up of the Middle West and North West was accompanied by the development of a mass-production type of agriculture conducive to large-scale operations and mechanization which was in sharp contrast to the diversified types of conventional agriculture of the East and South. Farm mechanization began in the middle west in the 1890's when self-propelled steam engines weighing ten to fifteen tons were introduced for ploughing, thrashing, and other agricultural operations. Up to this time, such highway bridges as had been built were designed for live loads consisting of people, herds of cattle, and an occasional wagon of perhaps no more than three or four tons. But the farm engines changed all this. The need for moving these engines from farm to farm created the demand for better roads and stronger bridges. The growth of the automobile a little later also served to intensify this demand. Consequently, several of the mid-western states took the lead in organizing their state highway departments in the early 1900's.

From this time on until the Bureau of Public Roads was organized in 1919, highway bridges were sometimes designed for a single ten- or fifteen-ton farm engine or road roller; sometimes for a ten- or fifteen-ton farm engine together with a uniform live load over the entire bridge; and sometimes for either a ten- or fifteen-ton farm engine or a uniform live load of somewhere between forty to one hundred p.s.f. of roadway, which ever produced the greater stress. In general, the farm engine design loading consisted of two axles spaced about ten feet apart with two-thirds of the load on the rear axle and wheels spaced six feet apart laterally. Some variation of these design loadings obtained until the new standard H-truck loadings were introduced by the AASHO in about 1923.

Development of Present Design Loadings: The American Association of State Highway Officials, usually referred to as the AASHO, came into being shortly after the Bureau of Public Roads was created in 1919. The AASHO Committee on Bridges and Structures was organized in 1921 and immediately set about the enormous task of preparing standard specifica-

tions for highway bridges and defining the standard live loads to be used for their design. By 1923, the live load to be used for bridge design was defined by the AASHO as a standard truck consisting of two axles spaced fourteen feet apart with 0.8 of the load on the rear axle and wheels spaced six feet apart laterally. This, incidentally, is the same as the Standard H-truck prescribed for bridge design by present-day AASHO specifications. For example, an H fifteen truck is a fifteen-ton truck with twenty-four kips on the rear axle and eight kips on the front axle.

The work of compiling bridge specifications began in 1921 with the organization of the Committee on Bridges and Structures of the AASHO. Soon after this certain parts, and later all, of the standard specifications for bridges were available in mimeographed form. However, it was not until 1931, a little over twenty years ago, that the first standard specifications for highway bridges appeared in printed form. At this time highway loadings were of three classes; namely, H 20, H 15, and H 10. For short spans (less than sixty feet) one design truck would be placed in each lane at such places as to produce the greatest stress effects. By this time also the number of heavy trucks on the highway was sufficiently numerous to warrant considering the probability that more than one heavy truck would occur in each lane at the same time. So in anticipation of very much increased truck traffic in the future, the AASHO introduced what was called the "truck train loading" in the 1931 specifications for use in the design of bridges in excess of sixty feet in length.

This truck train loading consisted of one truck of the gross weight indicated by the loading class followed and/or preceded by a line of trucks of indefinite length, each having a gross weight of three-fourths of the weight indicated by the loading class. In this truck train there is one truck for each forty-four feet along each lane of the roadway; or, the front axle of each truck is thirty feet from the rear axle of the preceding truck and the rear axle of each truck is thirty feet from the front axle of the next following truck. For example, the truck train for the H 15 loading would consist of one H 15 truck preceded and/or followed by an indefinite number of H 11 1/4 trucks in each lane, spaced as indicated, and all trucks in adjacent lanes are considered to be headed in the same direction.

For design purposes, the use of such a truck train would necessitate many long and tedious calculations. To simplify the computations incident to design, therefore, the 1931 specifications also introduced a new type of loading, defined as "equivalent loading," that would produce approximately the same stress effects as the truck train. For example, the H 15 equivalent loading consists of a uniform load of 480 pounds per foot of traffic lane combined with a single concentrated load of 13.5 kips for moment (19.5 kips for shear) so placed as to produce maximum stress.

From time to time since the first printed specifications in 1931, the AASHO has issued four revised editions of its specifications. However, except for the introduction of the truck-tractor semitrailer design truck, known as the standard H-S truck, in the 1941 specifications and a few minor revisions pertaining to the use of the equivalent loadings, the present design loadings of the AASHO are substantially the same as those given by the 1931 specifications.

Concern Regarding Adequacy of Bridges: During World War II, both the numbers and the weights of heavy trucks using the highway had increased far beyond anything that had been

anticipated in the 1920's and 1930's when a major portion of the nation's highways had been built. The number of trucks had increased from about 1.1 million in 1920 to about 4.6 million in 1940 and Figure 1 shows how average vehicle weights have increased through 1949 as compared with a prewar year. Figure 2 shows for the United States the number of gross weights of 30,000 pounds or more, 40,000 pounds or more, and 50,000 pounds or more per 1,000 vehicles from the prewar years (generally 1936 or 1937 through 1949). The trend in frequency of these loadings has continued to increase until the present time at about the rate indicated through 1949.

Along with the tremendous increase in the number and weight of heavy trucks, there has been a similar rise in the frequency of heavy axle loads. This can be seen from Figure 3, which shows the number of axle loads of 18,000 pounds or more, 20,000 pounds or more, and 22,000 pounds or more per 1,000 vehicles for the years 1942 through 1949 as compared with a prewar year.

As a result of these trends in highway loadings, bridge engineers and other highway officials have become increasingly concerned with regard to the safety and load-carrying capacity of the nation's bridges and other highway structures. In an effort to shed some light on these matters a number of studies were undertaken by the Texas Engineering Experiment Station in cooperation with the Bureau of Public Roads concerning the various heavy truck types and loadings using the highways and their stress producing effects on highway bridges. These studies have resulted in the publication of several Engineering Experiment Station Bulletins which contain tables and charts by which the stress-producing characteristics of most any heavy vehicle on the highway can be quickly and accurately determined. The first of these is Bulletin 127, "Method of Converting Heavy Motor Vehicle Loads Into Equivalent Design Loads on the Basis of Maximum Bending Moments." The second and third of this series are Bulletins 131 and 135 on the basis of maximum shears and floor beam reactions, respectively.

AVERAGE WEIGHTS OF LOADED AND OF EMPTY TRUCKS AND TRUCK COMBINATIONS IN THE SUMMERS OF 1942-49 AND IN A CORRESPONDING PERIOD OF A PREWAR YEAR

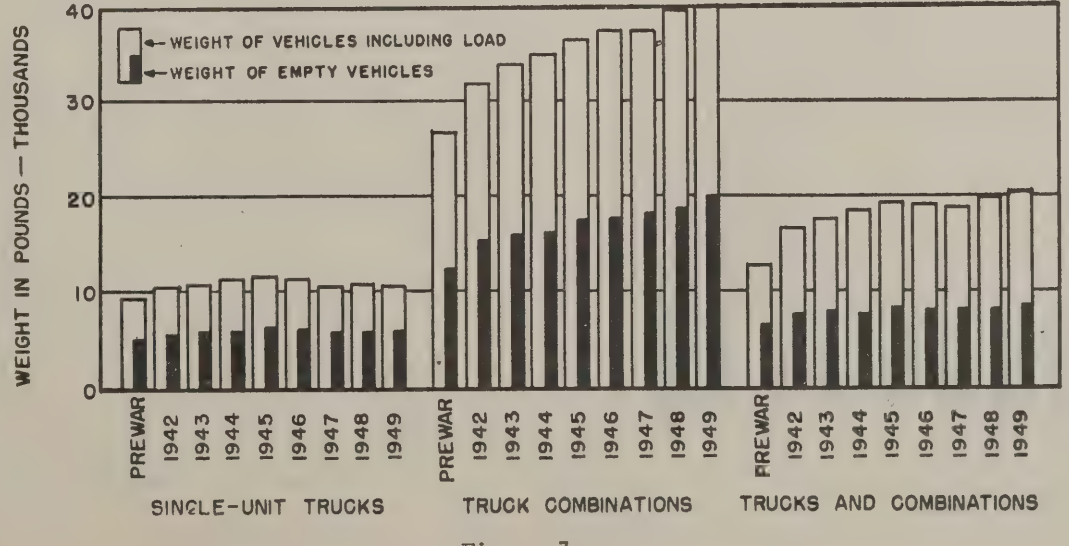

Figure 1

NUMBER OF HEAVY GROSS WEIGHTS PER 1,000 TRUCKS AND TRUCK COMBINATIONS (EMPTIES INCLUDED) IN THE SUMMERS OF 1942-49 AND A PREWAR YEAR

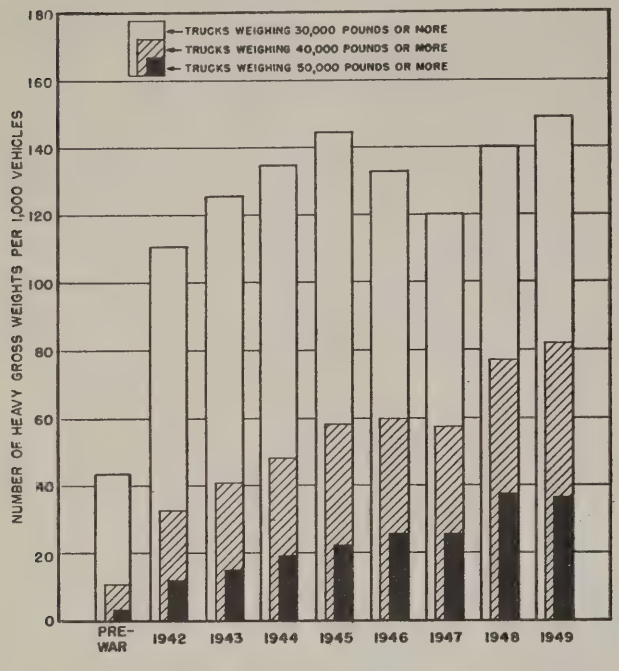

Figure 2

NUMBER OF HEAVY AXLE LOADS PER 1,000 TRUCKS AND TRUCK COMBINATIONS (EMPTIES INCLUDED) IN THE SUMMERS OF 1942-49 AND A PREWAR YEAR

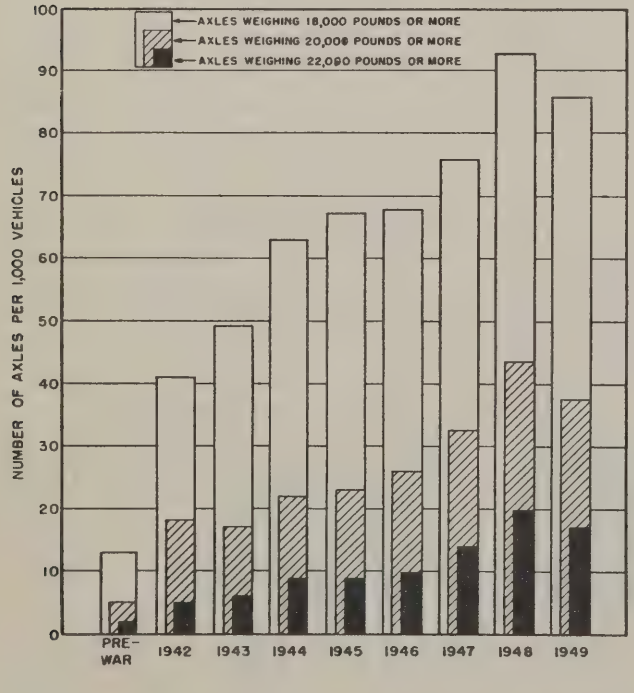

Figure 3

In addition to these studies, the author presented a paper at the annual meeting of the AASHO held in San Antonio, Texas, October 10-13, 1949, "Determination of Permissible Vehicle Weights on Bridges of H Loading Design," in which a rather simple method is developed for determining permissible vehicle loads. This paper is published in the AASHO Proceedings for 1949. In this paper it is shown that the maximum permissible weight on any group of axles, such that it would not produce more moment on any span than a standard H design loading of given designation, can be estimated rather accurately by use of the following equation (Eq. 1)

$$W = \left[\sqrt{C + La/4} + \sqrt{La/4}\right]^2$$

in which

W = Maximum weight in kips on any group of axles such that it will not produce more live load moment on any span than a standard H-design

AXLE - GROUP - LOADS ON BRIDGES OF H LOADING DESIGN

GRAPH SHOWS MAXIMUM PERMISSIBLE WEIGHTS ON ANY GROUP OF AXLES WHICH, FOR ANY NORMAL DISTRIBUTION OF LOAD, WILL NOT PRODUCE MORE MOMENT ON ANY SPAN THAN THE H LOADING INDICATED

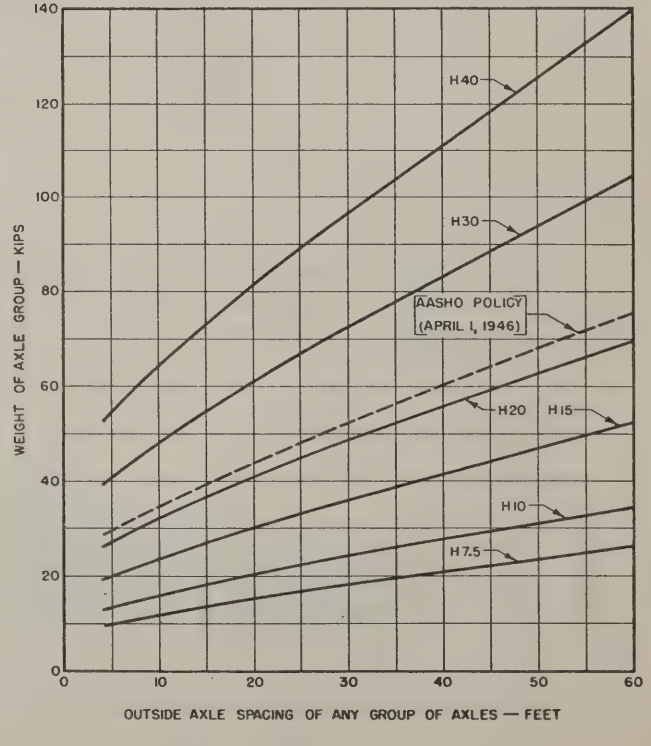

Figure 4

loading corresponding with the lane loading constants, C and a, in Equation 1.

L = Distance in feet between the extremes of any group of axles.

C = Concentrated load (for moment) in kips corresponding to H-lane loading designation under consideration.

a = Uniform load in kips per foot corresponding to H-lane loading designation under consideration.

Note: If pounds instead of kips are used for the constants C and a in Equation 1, the weight, W, will also be in pounds.

The permissible axle-group loads indicated by Equation 1 for bridges of given H-loading design are shown graphically in Figure 4.

In Figure 4 it will be noted that the maximum permissible axle-group loads recommended by the AASHO Policy of April 1946, correspond approximately to an equivalent H 21.5 truck. In connection with Equation 1, it will also be noted that the length of bridge span does not enter into the determination of permissible axle-group loads. The only things that do enter into such determinations are the wheel base L of the axle-group or vehicle and the constants C and a, corresponding to the H-loading design designation under consideration. Incidentally a brief discussion of Equation 1 and Figure 4, together with a table of values corresponding with Figure 4, are given in Engineering Experiment Station Bulletin 127, mentioned above.

Highway Loads on H 15 Bridges: The vast majority of existing highway bridges in the United States were designed and built in accordance with standards which correspond approximately with present H 15 design of AASHO. The replacement cost of these bridges would probably be no less than 8 or 9 billion dollars, and for this reason these existing bridges are not likely to be replaced for many years to come. Therefore it is extremely important that all interested parties understand and appreciate the loading limitations indicated for bridges of H 15 design.

In response to this need the Engineering Experiment Station developed a method for determining the degree of overstress or understress produced by equivalent design loadings on bridges of given design designation. These studies resulted in the publication of Bulletin 132, "Stress Producing Effects of Equivalent Design Loads on Modern Highway Bridges," by Henson K. Stephenson and Kriss Cloninger, 1953. This bulletin shows that the degree of overstress or understress indicated by the design stress ratio X, or per cent of design stress, can be determined by a very simple straight line equation. Bulletin 132 also presents a rather complete study of the maximum bending stress effects produced in simple span deck girder bridges of H 15 design by six heavy vehicle types, each representing the average of the ten heaviest vehicles of the type among those reported by the nation-wide loadometer survey of 1942. The sizes, weights, and other data pertaining to these six heavy truck types are as follows:

Truck Type	No. of Axles	Wheel base feet	Gross Weight pounds
2	2	14.8	35,306
3	3	18.8	49,673
2-S1	3	32.2	65,359
2-S2	4	35.7	63,266
3-S2	5	40.2	75,650
3-3	6	52.3	86,380

The bridges selected for this study consist of a concrete deck of minimum thickness supported by unencased steel beams which are so spaced that the maximum live load bending stress produced in an interior stringer by a single vehicle in one lane only will amount to 75 per cent of that produced by identical vehicles in each lane simultaneously. The reason for selecting this type of construction is because the ratio of dead-load stresses to total design stresses is smaller than would obtain for any of the heavier types of construction. Therefore any conclusions arrived at concerning the stress-producing effects of a given vehicle on any particular bridge will tend to be on the conservative rather than the unsafe side.

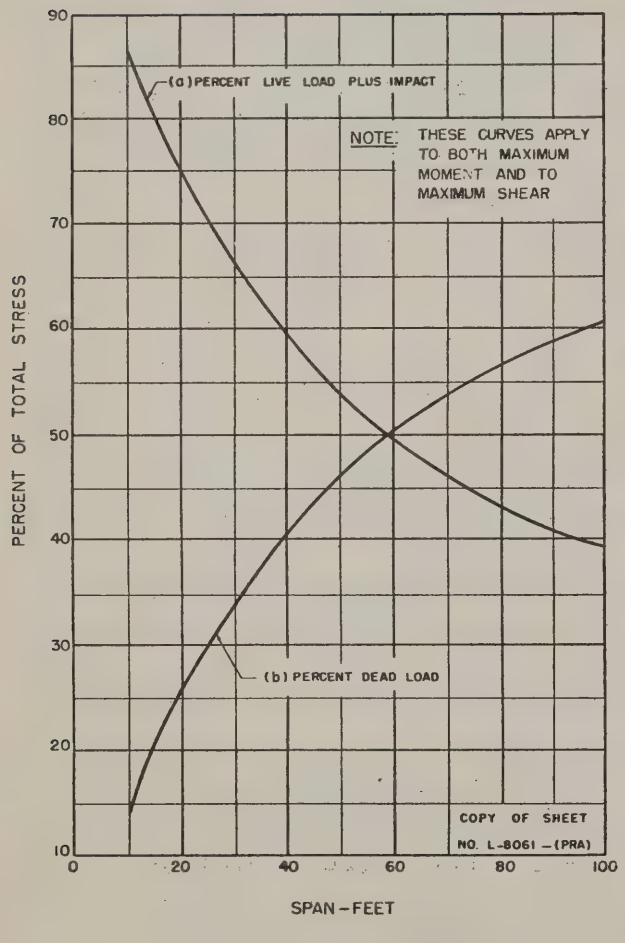

Figure 5

For this minimum type of light construction, the ratio of dead-load stress to total design stress and the ratio of live load plus impact stresses to total design stress for each span of H 15 design up to 100 feet in length are substantially the same as those indicated by the curves given in Figure 5.

With the information given in Figure 5, it is explained in Bulletin 132 how the per cent of design stress produced by each of the above-mentioned six heavy trucks on spans up to 100 feet is determined for each of the four loading conditions, defined as follows:

Case 1: One vehicle in each line with full allowance for impact.

Case 2: One vehicle in one lane with full allowance for impact.

Case 3: One vehicle in each lane with no allowance for impact.

Case 4: One vehicle in one lane with no allowance for impact.

The per cent of design stress produced by each of the six heavy trucks on each span length corresponding to Cases 1, 2, 3, and 4 is given by Figures 6.1, 6.2, 6.3, and 6.4, respectively

In Figures 6.1 - 6.4, inclusive, it will be seen that under the most severe conditions of loading – that is Case 1, when one vehicle is placed in each lane in exactly the proper position for maximum moment and with full allowance for impact – the maximum overstress produced by any of these vehicles on any span is 32 per cent. This maximum overstress of 32 per cent would be produced on a

STRESS PRODUCING CHARACTERISTICS OF SIX HEAVY VEHICLE TYPES REPRESENTATIVE OF HEAVIEST VEHICLES REPORTED BY THE 1942 LOADOMETER SURVEY

Case 1: One vehicle in each lane with full allowance for impact.

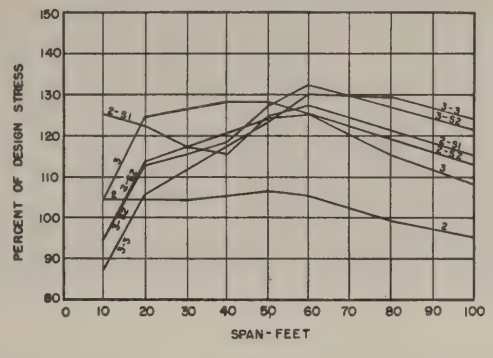

Figure 6.1

Case 2: One vehicle in one lane with full allowance for impact.

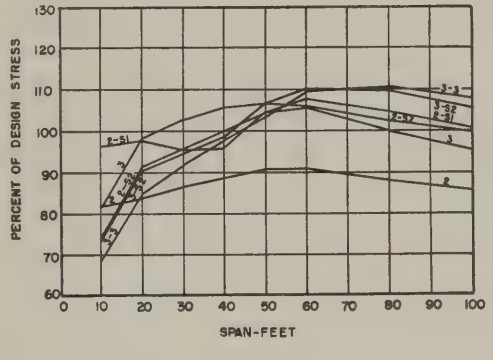

Figure 6.2

STRESS PRODUCING CHARACTERISTICS OF SIX HEAVY VEHICLE TYPES REPRESENTATIVE OF HEAVIEST VEHICLES REPORTED BY THE 1942 LOADOMETER SURVEY

Case 3: One vehicle in each lane with no allowance for impact.

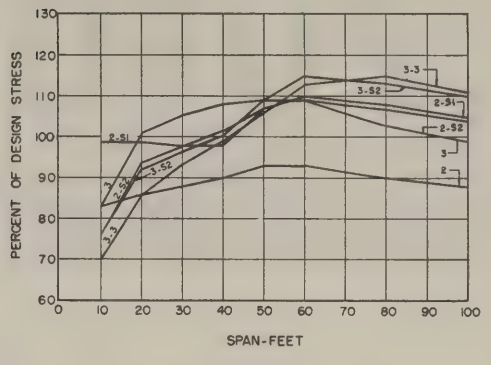

Figure 6.3

Case 4: One vehicle in one lane with no allowance for impact.

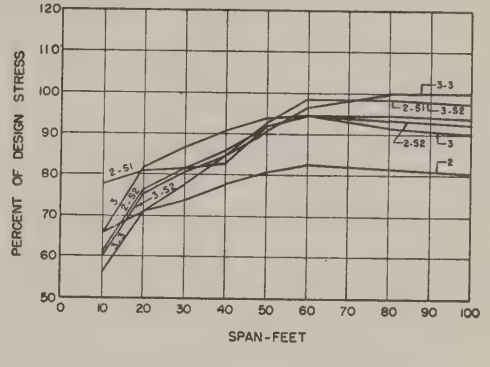

Figure 6.4

60-foot span if one of the Type 3-S2 trucks were placed in each lane simultaneously with full allowance for impact. For Case 2, however, with one vehicle in one lane only with full allowance for impact, it will be noted that the overstress nowhere exceeds the 11 per cent overstress which obtains for the Type 3-S2 truck and the Type 3-3 truck on the 60 and 80 foot spans, respectively. Case 2, incidentally, is believed to represent approximately the most severe condition of loading likely to be encountered under ordinary traffic conditions. Actually though, under ordinary traffic conditions on main rural highways, with pneumatic tires and smooth roadway, it would seem logical to assume that the actual stresses produced would be somewhere between those indicated for Case 2, with full allowance for impact, and Case 4, with no allowance for impact.

Owing to the fact that the AASHO specifications permit stresses equal to 150 per cent of normal design stresses in H 15 bridges to accommodate an unusually heavy single load occasionally, it will be seen that none of the six heavy trucks investigated would produce stresses in an H 15 bridge that would be considered critical even under the most severe loading conditions indicated in Case 1, Figure 6.1. A careful study of the sizes and weights of these six heavy trucks and the stresses they produce in H 15 bridges will undoubtedly generate considerably more confidence in the serviceability and load-carrying capacity of bridges of H 15 design.

FUTURE OF THE

With this Twenty-eighth Annual Highway Short Course we are beginning to follow a plan for our program which is materially different from that of recent years. The group session plan, which we are using this year, is a start along the road which we hope to travel during the coming years.

Early in the spring of this year a group from the College met with officials of the Highway Department in Austin. At this meeting the possibilities and shortcomings of our Highway Short Course were thoroughly discussed. As a result it was decided to change the type of program and the emphasis in the hope that those who attended would obtain more value from the Short Course program. This program which we have begun today is the first of the type settled upon. This program is admittedly a trial program and we hope that all of you will tell us what you do or do not like about it. Any suggestions that you have for improvement of future programs will be most welcome. It was further decided that in our future programs we would utilize the services of the college staffs of this and other colleges, people from contractors and materials organizations, and those from outside the state particularly qualified in various fields as well as many people from our State Highway Department. With the return of the Short Course to the spring of the year in 1955 we also will try to secure a maximum of student attendance at the programs with the thought that this will be of mutual benefit to all concerned. We hope by so doing to create additional interest in the highway field among the students and to give them an opportunity to mingle with the practicing engineers of the state.

We have two major objectives for the Short Course program in the future. First we want to give the program just as much educational value as we possibly can for the time allotted to it. Much can be learned in three or four days time when the

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Civil Engineering Department
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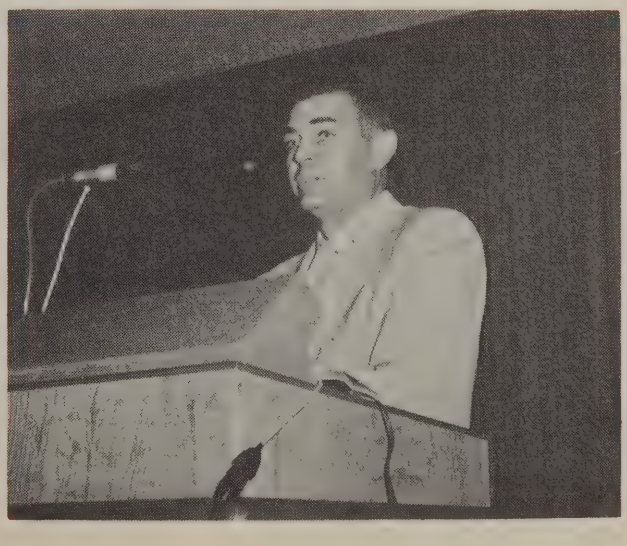
College Station, Texas

engineer is free from the problems of his ordinary work day and can concentrate on a subject in which he has a vital interest. Secondly, we want to get just as much individual participation in the discussion of the subjects under study as we possibly can. There are many good ideas tucked away in the back of your minds which you have been hesitant to present. We want to break down any misgivings that you have and get these ideas into the open so that others may benefit from them.

In order to have an educational program we must have a popular program, in other words, one that you men will want to come to. The good fellowship which you have here, the opportunity to renew acquaintances and to discuss problems of mutual interest are very valuable features of this Short Course. If the discussions here can bring you new developments and new ideas in your major field of interest and if, in addition, we can also incorporate your thoughts the program will be more valuable to you as individuals and to highway construction in this great State of Texas.

Development of a program for each Short Course will not be an easy matter. The cooperation of all interested people must be obtained if we are to have the type of program which I feel all of us want. You men must let those in charge of program arrangements know what you are interested in, where your problems lie and what phases of highway engineering you need to know more about. It is my thought that the program for each field of technical interest should be developed by and for the people in that field. For example, if we have a group from the Bridge Division who desires to study the matter of design of prestressed bridges or bridge foundations they should have an opportunity to do just that. Another group from your expressway offices



might want to study the design of entrance and exit ramps. Still another group might want to study methods for field control of compaction of subgrades and base courses. In our program arrangements opportunity will be provided for this to be done and the College in cooperation with the Highway Department will try to get the best available talent to instruct and lead the discussions. There has been some indication that the Short Course program may be valuable to the contractors of the state. Our Engineering Extension Service has had conversations with two contracting organizations regarding the possibility of providing technical training to some of their supervisory personnel in the fields of bituminous pavements and soil mechanics. It is entirely possible that the demand for this type of service can be met as a part of the program of this Short Course. We are very much interested in any thoughts which our highway contractors have concerning subjects for our program which will be of most interest to them. It is our desire to weld all elements of the highway industry, design, construction, and maintenance into the program. This will give the best balance and should yield the most valuable program for all concerned. What we need from the field in order to prepare the best possible program is a continual flow of ideas along with a bit of tolerance if we are not able to work your thoughts into the first available program. Do not hesitate to write at any time and give us your thoughts.

The participation phase, our second objective, is entirely up to those of you who attend the Short Course. Many engineers, and I am as guilty as anyone, are hesitant to speak out in a group of more than two or three. This is bad for the individual and bad for the profession. In all lines of engineering activity we need more vocal activity by those who are engaged. If you don't agree with what has been said, stand on your feet and say so as an engineer and a gentleman. If you can add something to what has been said, do so. With the smaller group meetings we are trying to give all a chance to be a part of the Short Course program and sincerely hope that most of you will do so willingly. We want the young fellows to sow their wild ideas and the older experienced heads to squelch them if they can. We believe all will learn in the process.

Many things of value can come from a program such as this. In the program this time we are going to talk about Joint Seals for Concrete Pipe Culverts. It has never occurred to me that this particular operation warranted further study. A little discussion with several whom I knew indicated that I was wrong. There is some weight of opinion to support the idea that there may be better and cheaper ways of doing this job than that called for in the Highway Department's Standard Specifications. Out of a study and discussion of the various methods in use may come a recommendation for a change in the specifications which will save the State some money and allow you to build or rebuild a few more miles of road per year. The greatest value, however, lies in the possible benefits which may accrue to the individual. It has always been my thought that the people in an organization make that organization. For building and maintaining highways we have here in Texas the finest Highway Department and the finest group of Contractors. In this Short Course we can add a little to the knowledge of our people, we can make definite advancements, and we will be better.

Our future plans anticipate that the Short Course will open on Monday or Tuesday with a general session covering topics of wide interest to those engaged in the highway field just as we have done here today. We will then have three or four days of individual technical sessions which may be devoted to almost any subject under the sun which is important to highway people. The facilities and laboratories of the College will be available for these technical programs so that almost any type of instruction or group study will be possible. We want to make this Short Course a real part of the highway program for this State.

WEDNESDAY, SEPTEMBER 8

All meetings in Memorial Student Center.
Rooms are shown by name or number.

PAVEMENTS SECTION

Morning: Ballroom

Chairman: Spencer J. Buchanan, Professor
Soil Mechanics and Foundation Engineering
A & M College of Texas

9:00 Compaction, Principles, and Relation to Strength
W. H. Moore, Senior Research Engineer
Texas Highway Department
Austin, Texas

J. E. Hunt, Soils Engineer
Texas Highway Department
Austin, Texas

11:00 Use of Heavy Pneumatic Rollers
Professor Spencer J. Buchanan
A & M College of Texas

Afternoon: Ballroom

Chairman: Professor Spencer J. Buchanan

1:30 Triaxial Procedure for Flexible Pavement Design
Chester McDowell, Senior Soils Engineer
Texas Highway Department
Austin, Texas

3:00 Design of Cross Section as Influenced by Edge Failure Problem
A. C. Kyser Jr., Assistant District Engineer
Texas Highway Department
Houston, Texas

RELATION BETWEEN COMPACTION AND

W. H. Moore, Senior Research Engineer
Texas Highway Department
Austin, Texas

It is clearly recognized that no laboratory investigation can duplicate exactly an actual construction project. The conditions that exert critical influence on the outcome of soils operations are susceptible of control to a very considerable degree in the laboratory, but in actual construction such control taxes to the utmost the skill and the ingenuity of the field engineer or the builder. Nevertheless, the very fact that the variables can be controlled in the laboratory makes it possible to examine them one by one, and to reveal the underlying principles. It is the hope of the writer to call to your attention - or more accurately to recall to your attention - these underlying principles, complicated only by a minimum of unavoidable technicalities.

This discussion contains nothing new except a new look at some old facts.

The previous speaker, Mr. J. E. Hunt, has described the compaction of soils. Let us begin with a definition of stability. For the purpose of this discussion, the stability of soil is its ability to resist, over a relatively long period of time, cumulative or permanent deformations resulting from loads superimposed by surcharge or by highway traffic. How shall we measure stability?

It is essentially true that a soil structure, loaded beyond its ultimate strength, actually fails by shearing

STABILITY OF SOILS

along some interior surface or surfaces. Its stability depends upon the resistance to shear developed along the most dangerous surface, under the particular conditions of the loading. It is not accurate to say that a soil has a shearing strength. It has just as many shearing strengths as there are degrees of pressure perpendicular (or normal) to the plane of shear. Most of you probably are entirely familiar with shear diagrams, but for the benefit of those who have not been using them recently, it may be well to review the subject briefly. It is easier to visualize the direct shear test than other methods. The first figure represents such a test.

The apparatus consists essentially of a rectangular metal box cut in two horizontally, the lower half being fixed rigidly while the upper half is attached to a horizontal pulling mechanism and a load-measuring device. A second loading device applies a selected, constant load normal to the plane of shear, that is, vertically. The soil specimen completely fills the box. The horizontal shearing load is increased until the specimen fails. The test is repeated with new specimens at several different normal stresses; that is, perpendicular to the plane of shear. The results are plotted as shown in the figure. All are highly idealized for purposes of illustration. Figure 1a is typical of a clean, sandy material with no cohesion, and no strength unless confined by some normal pressure. The internal friction produces an increase in shearing resistance as the normal stress is increased. Figure 1b illustrates a very fat clay material having cohesion which, theoretically, is an inherent quality not affected by pressure normal to the shear plane, and having no internal friction to increase the shearing resistance as the normal stress increases. Actually, a material with zero friction is rare. Figure 1c illustrates a material having both cohesion and friction, such as a good sandy clay soil.

Another method of determining the shear diagram, namely, the triaxial compression test, is employed by those who feel that the rigid shear box has a large, adverse effect on test results. In the triaxial test a constant, all-around pressure is applied to the specimen through a rubber diaphragm, at the same time, a constantly increasing vertical pressure is applied until failure occurs. By testing a series of nearly identical specimens at various different lateral pressures, and plotting the test results in Mohr's diagram of stress, the shear diagram or "rupture envelope" may be obtained.

The upper portion of Figure 2 illustrates Mohr's diagram of stress, a graphical procedure worked out many years ago by Professor Otto Mohr. The mathematical derivation of the figure has been checked thousands of times and need not be discussed here. For our present purpose it is sufficient to consider how it is constructed and what it indicates. On the horizontal axis mark the points that represent the lateral pressure and the ultimate vertical strength of one test specimen. With the distance between these points as a diameter, construct a circle (in this case a semi-circle will be enough). Each point on the circle represents the shear stress and the normal stress at the instant of failure on the planes which make some particular angle with the direction of the major principal stress (in this case the vertical stress). The point at which the ultimate vertical stress was plotted represents the plane of the major principal stress.

The point directly above the center of the circle represents a plane making an angle of 45 degrees with the plane of the major principal stress, and the point at which the minor principal stress (lateral pressure) was plotted represents a plane tilted 90 degrees to the plane of the major principal stress. There is one plane on which the shearing resistance barely equals the shearing stress, indicated by the outer end of the dotted radius. In a similar manner, stress circles are plotted for each specimen tested. Draw the common tangent to the circles and you have the shear diagram for the material tested. If a large number of tests resulting in ultimate failure are made and a large number of stress circles are drawn, a sort of envelope is formed. This envelope is called Mohr's line of rupture and it shows the relation between the ultimate shearing strength and the normal stress at the instant of failure on any plane. In the previous figures, for simplicity of illustration, we have shown the idealized, straight line rupture envelopes which conform to Coulomb's hypothesis. In actual practice, the rupture envelope is seldom straight and Mohr's line of rupture, which does not necessarily conform to Coulomb's hypothesis, is realistic. There are three points to be noted:

1. Any combination of normal stress and shearing stress which would be plotted above the line of rupture would cause failure of the material.

2. Any combination of normal stress and shearing stress which would be plotted below the line of rupture would not cause failure.

3. The most dangerous plane is not necessarily the plane on which the greatest shearing stress occurs for this plane usually has an excess of shearing resistance. Refer to upper Figure 2. The plane on which shearing stress equals shearing resistance is the most dangerous condition.

To return momentarily to our idealized illustrations, refer to Figure 3. The rupture envelopes of the typical materials, determined by triaxial compression tests and Mohr's diagram of stress would appear as indicated.

The combined stresses, that is, major and minor principal stresses, which occur at each of a series of points on a given level within a soil mass, as a result of applied loads, may be estimated mathematically. Stress circles representing these combinations of stress may be plotted for all points on a given level, and one may draw an envelope which encloses all of them and which represents the upper limits of the applied combinations of stress. The shape of such envelopes is similar to the fine lines in the shear diagrams which will follow.

We have waded through a slough of technicalities but perhaps it was worth it for we have come out with a criterion by which we can judge the stability of soils, namely, the rupture envelope.

The soil specimens used in these tests were compacted with certain compactive efforts known to be obtainable in construction, then were subjected to dry curing (except cracking soils) and finally were subjected to capillary absorption under conditions of restraint that closely resemble field conditions. Specimens were tested only after they had reached an equilibrium with their environment.

Consider Soil No. 39-7-MR, a very sandy soil from Austin County. Five different compactive efforts were used to determine the moisture-density curves shown in Figure 4. The lowest compactive effort, 3.98 foot pounds per cubic inch of specimen, is approximately equal to six-tenths of the standard A. A. S. H. O. compaction. The others are about 1, 2, 3, and 4.4 times the standard A. A. S. H. O. compactive effort. Obviously, we do not continue to obtain equal increases in density for equal increases in compactive effort. But what of the stability of the soil at these various conditions?

Figure 5 shows Mohr's diagram of stress for the largest (dotted lines) and the smallest (solid lines) compactive efforts. Intermediate compactions were omitted from the diagram to avoid crowding. All of the rupture envelopes were then transferred to a shear diagram (Figure 6) which is

essentially the classification chart used in flexible base design. The soil class numbers are the same. It will be noted that there is a fairly steady increase in stability for each increase in compactive effort. Each rupture envelope is higher than the previous one. At the greatest compactive effort this soil, which contains no coarse aggregate, is within the lower portion of the class usually described as second class base material. There was no measurable swell as the specimens absorbed capillary moisture.

A similar procedure has been followed with Soil 47-934-E, a good, sandy soil from Goliad County. Only three compactive efforts were employed, equivalent roughly to 1, 2, and 3 times the standard A. A. S. H. O. Figure 7 shows the three moisture-density curves, again with diminishing returns so far as density alone is concerned. Omitting the Mohr's diagram we go directly to the rupture envelopes representing specimens from the peaks of the three compaction curves. See Figure 8. There has been continuous improvement in shearing strengths at all normal stresses as compaction has increased. Again, there was no swell due to absorption.

The next soil tested was No. 47-935-E, a sandy loam soil from Goliad County. The moisture density curves are shown in Figure 9. The density-for-compactive-effort picture is about the same. However, Figure 10 shows a departure from the response of the previous soils. At the highest compactive effort there was a considerable absorption of capillary moisture and measurable volumetric swell. The failure of shear strength to increase makes the highest compactive effort uneconomical.

Soil 47-936-E is slightly more plastic than the previous soil. Figure 11 shows the three moisture-density curves. Figure 12 reveals that a slight increase in stability resulted from the greatest compactive effort in spite of a little volumetric swell. Some desirable characteristic of this soil must have compensated for the swell. However, the improvement is not of a large order.

Let us now consider a truly plastic, heavy, clay soil. Lab. No. 39-11-MR represents the widespread black waxy soil of Texas (technically, it is Houston Black Clay). Tests at low compactive efforts soon revealed the nature of this material so that it was unnecessary to attempt higher compaction. Figure 13 shows three groups of specimens compacted at three different compactive efforts, the highest of which is slightly greater than standard A. A. S. H. O. compaction and the next lower compactive effort is slightly less than standard A. A. S. H. O. The lower right corner of the graph shows the condition of these specimens after a 60-day period during which they had reached an equilibrium with capillary moisture. The swell which accompanied absorption brought about a sharp reduction in final density. The rupture envelopes in Figure 14 indicate that the middle degree of compaction was worth while but the swell resulting from the highest compaction was destructive enough to reduce the final shearing strengths.

Figure 15 is a recapitulation. For this graph the shearing strength at 20 p. s. i. normal stress was selected because this seems to be the most critical condition for these soils. This particular shearing strength is plotted against the compactive effort necessary to produce it for each soil.

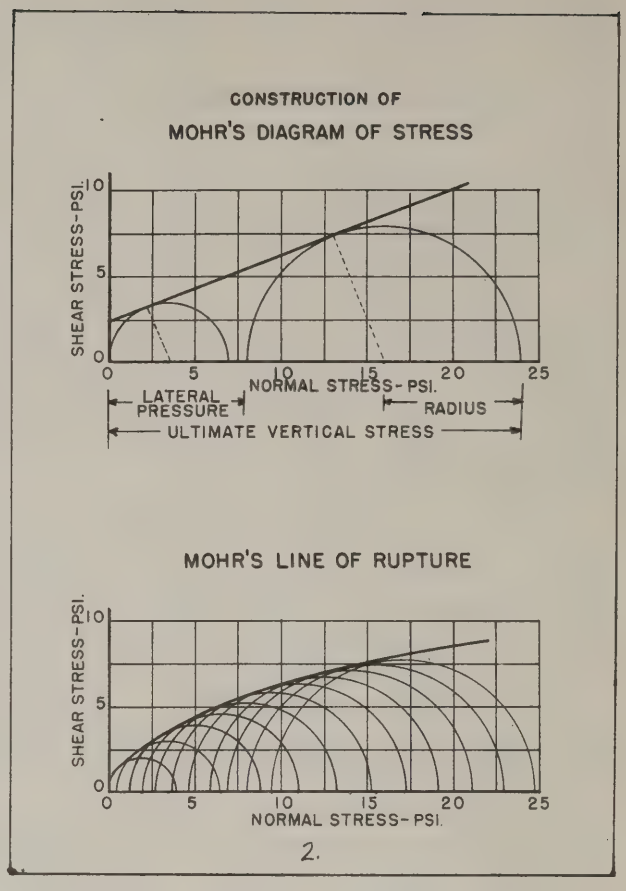
It seems to be reasonably clear that increased stability resulting from higher compaction is profitable if the stability is permanent. Such increased stability in foundation soils should increase the life span of the superstructure, and should permit the use of thinner surface courses. How permanent, then is this increased stability? What affects its permanence?

Influences tending toward permanence are low moisture demand by the soil, protection from surface weathering, and heavy surcharge. The first two influences are fairly obvious. The third is of critical importance. No matter how active a soil may be, it can be restrained from swelling by a sufficient surcharge. In most highway structures such a surcharge usually is not possible. A fourth influence should be set apart from the others. Work is

required to densify a soil. The particles are wedged together, not simply compressed like a spring. They will not fly apart when the load is removed. In the absence of swell, a large quantity of work is required to destroy the compacted structure of a soil. It can be very stable.

Adverse influences are high moisture demand, swell, weathering, and dry-weather cracking. Little can be done about the first two unless a very heavy surcharge can be applied. Weathering for the most part may be reduced by blanket layers. The writer is unable to suggest any satisfactory way to prevent dry-weather cracking.

It seems reasonable to conclude that good, sandy soils, or even slightly loamy soils, will yield good increases in stability for increases in density, even beyond those densities which we now obtain. Active clay soils should be densified only to the extent that their condition can be maintained.



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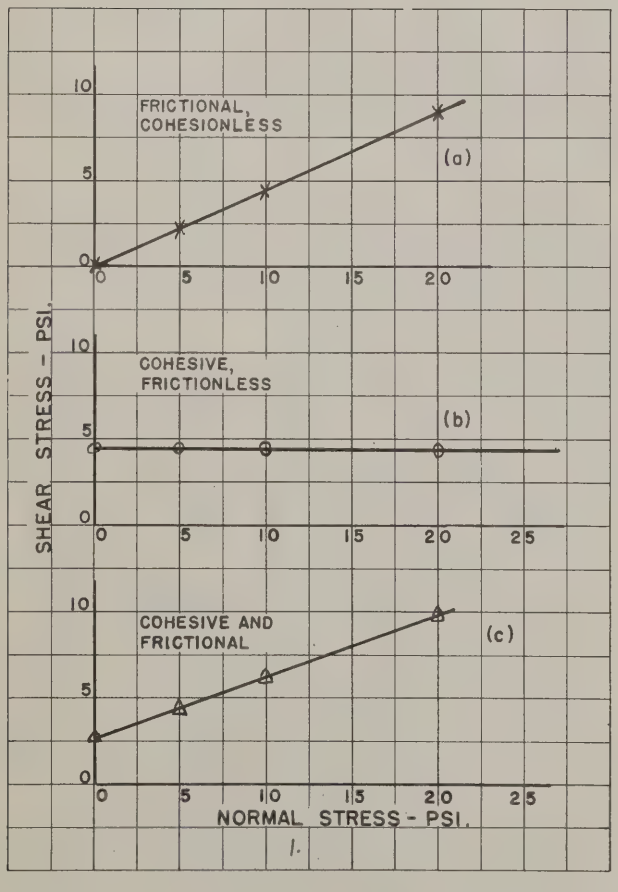

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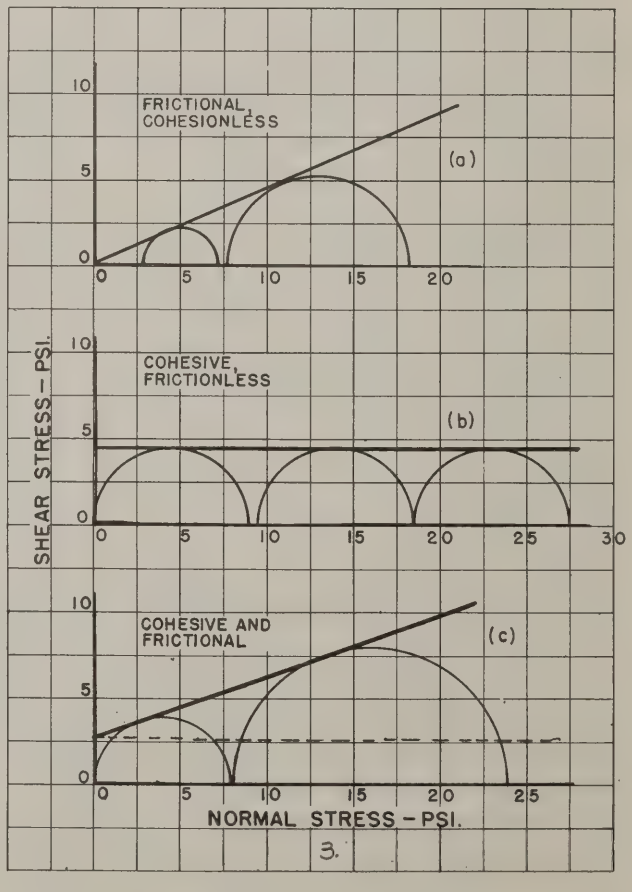

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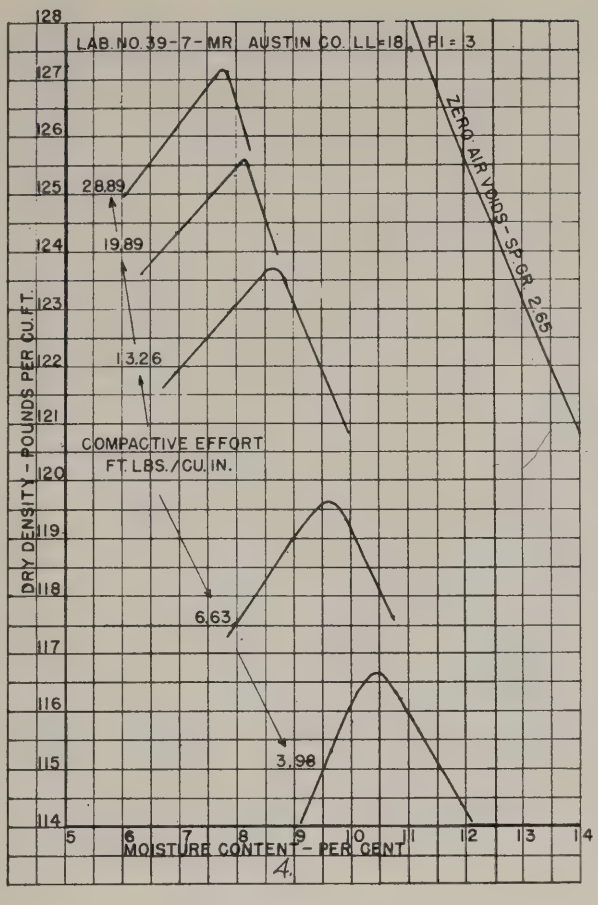
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DRY DENSITY - POUNDS PER CU. FT.
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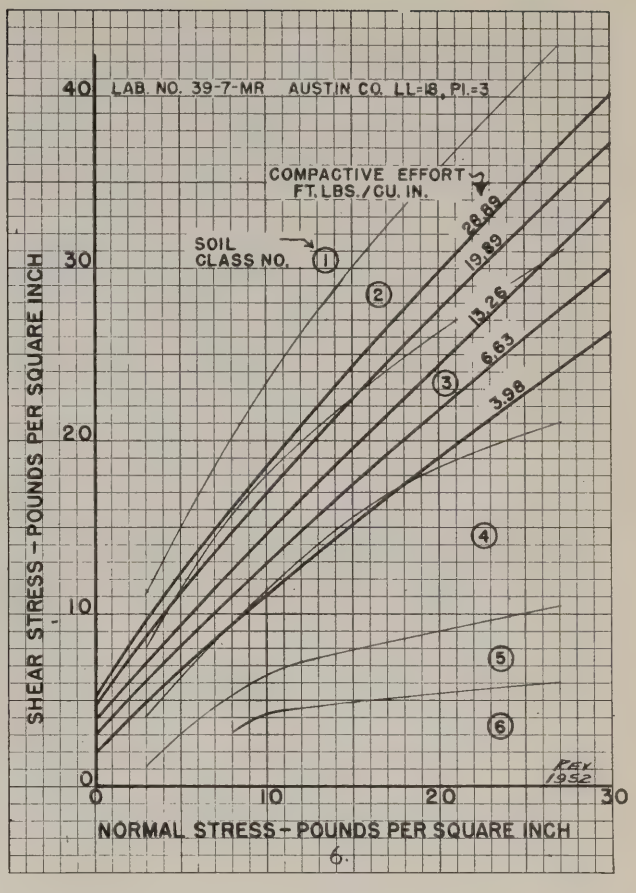
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19.89
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NORMAL STRESS - POUNDS PER SQUARE INCH
REV 1952
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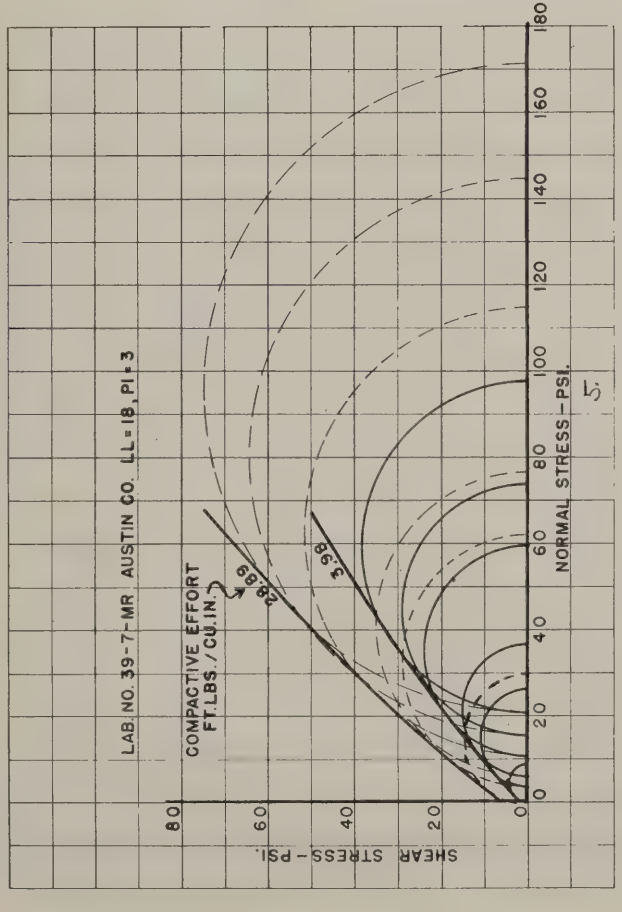
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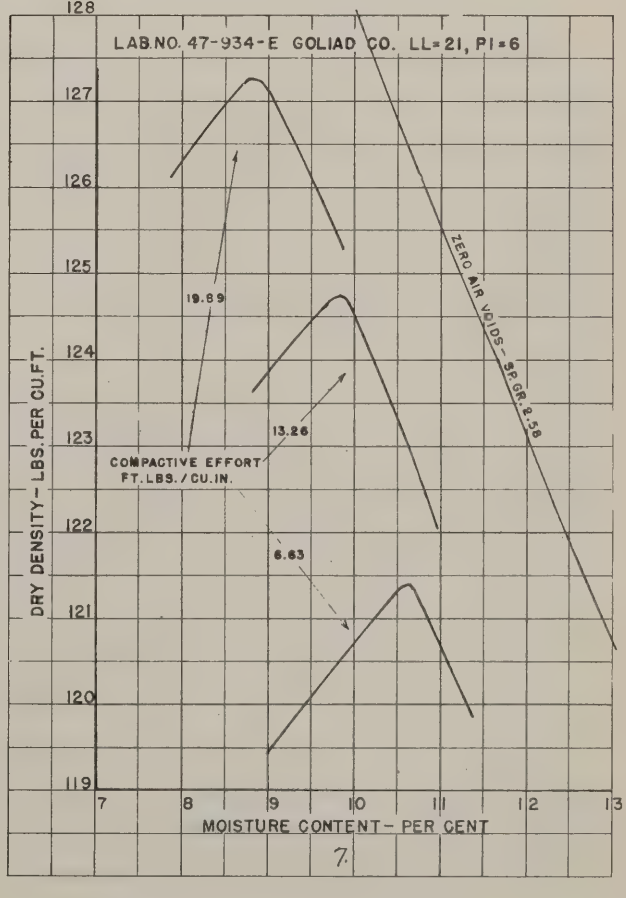
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DRY DENSITY - LBS. PER CU. FT.
MOISTURE CONTENT - PER CENT
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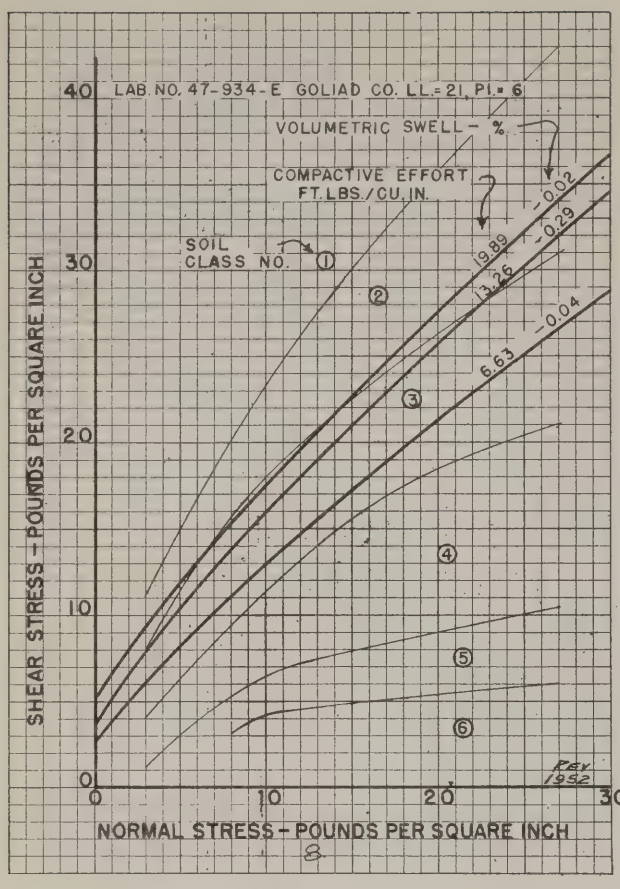
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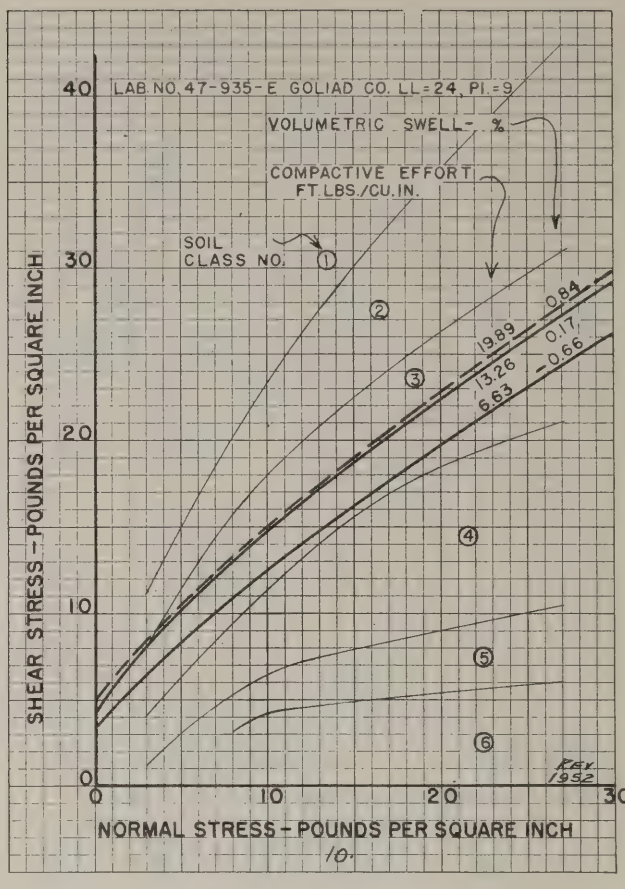
NORMAL STRESS - POUNDS PER SQUARE INCH
Rev 1952

8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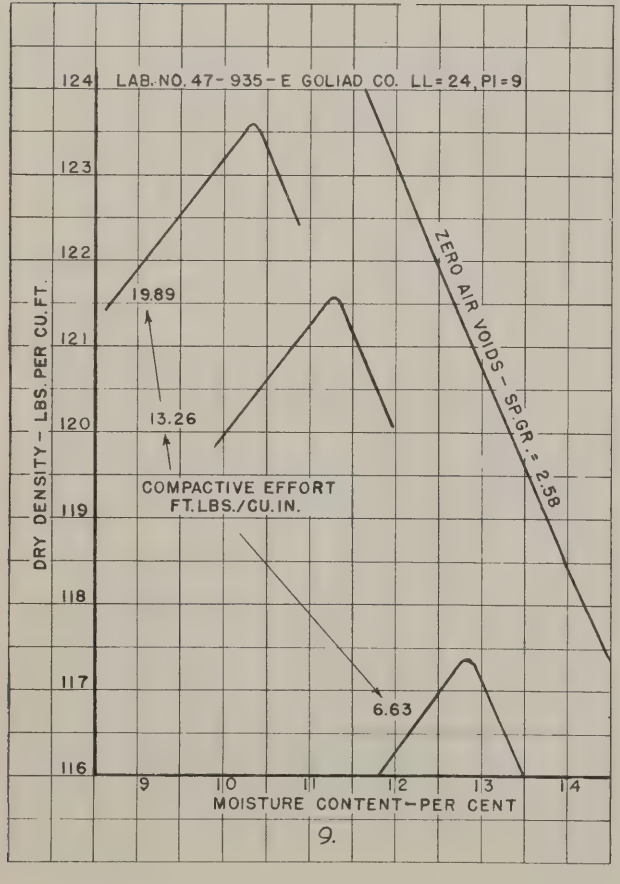
10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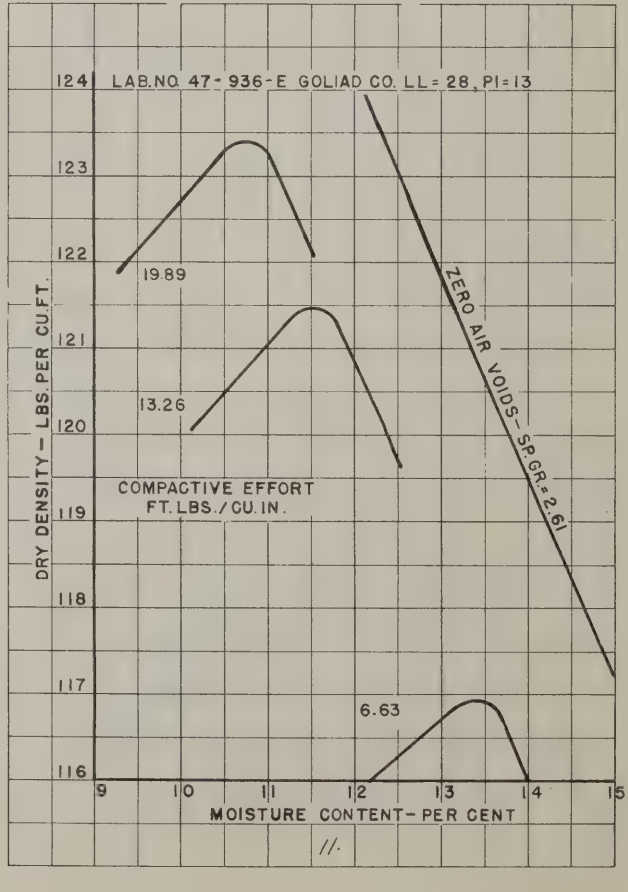
DRY DENSITY - LBS. PER CU. FT.
MOISTURE CONTENT - PER CENT

9.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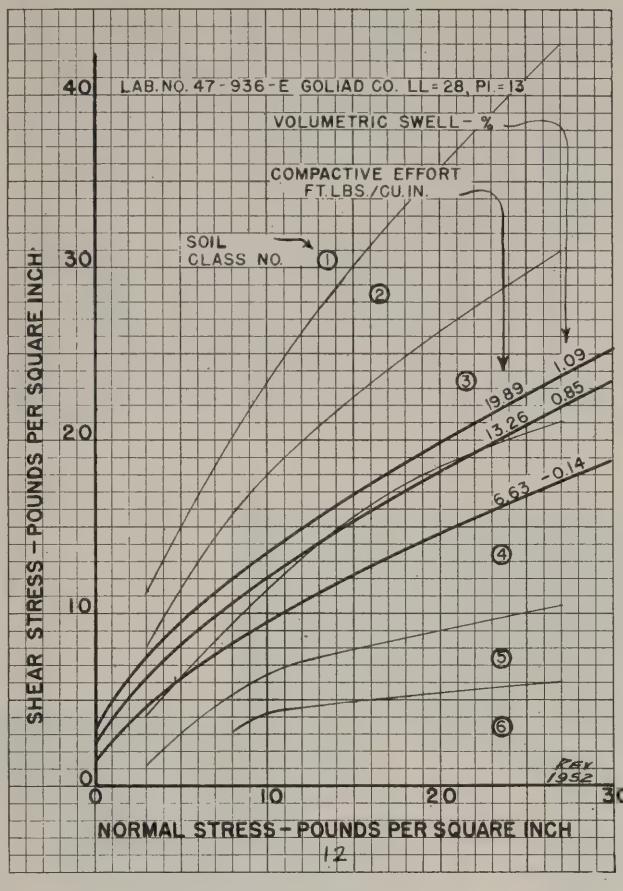
MOISTURE CONTENT - PER CENT

11.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0.85
13.26
-0.14
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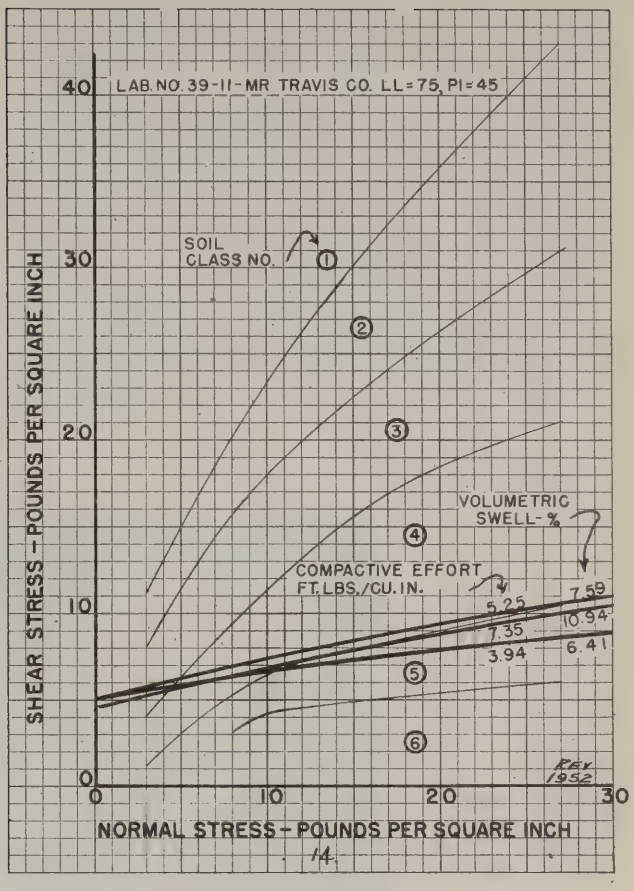
SHEAR STRESS - POUNDS PER SQUARE INCH
NORMAL STRESS - POUNDS PER SQUARE INCH
REV 1952
0
10
20
30
40

12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10.94
7.35
6.41
3.94
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NORMAL STRESS - POUNDS PER SQUARE INCH
REV 1952
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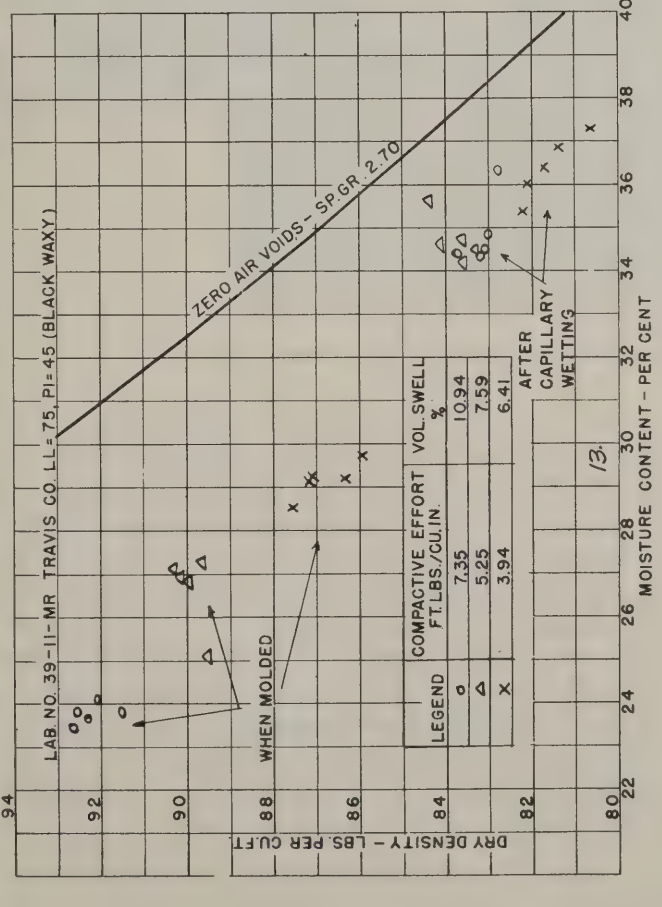
10
20
30
40

14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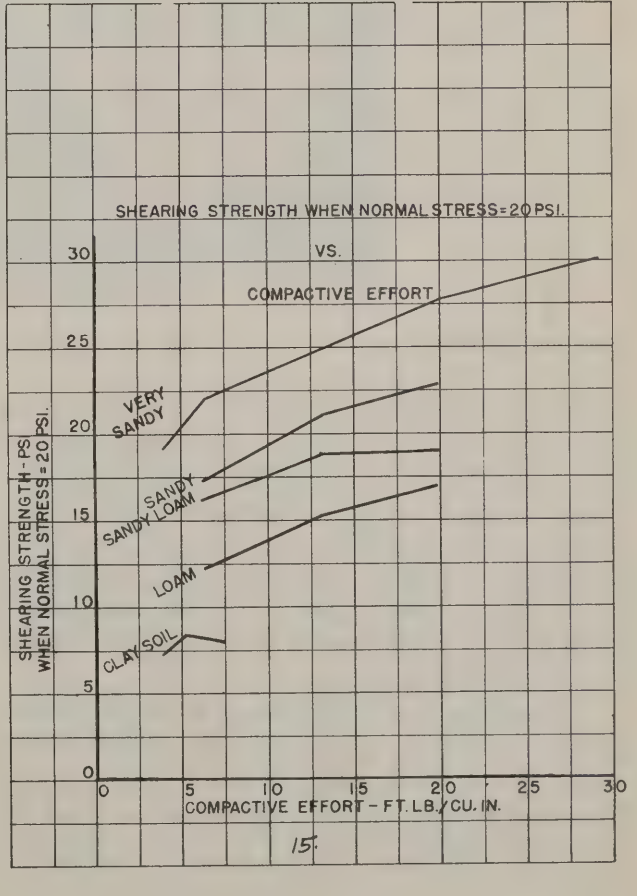
AFTER CAPILLARY WETTING
LEGEND
COMPACTIVE EFFORT FT. LBS./CU. IN.
VOL. SWELL %
7.35
10.94
5.25
7.59
3.94
6.41
MOISTURE CONTENT - PER CENT
DRY DENSITY - LBS. PER CU. FT.
94
92
90
88
86
84
82
80
22
24
26
28
30
32
34
36
38
40

13


SHEARING STRENGTH WHEN NORMAL STRESS = 20 PSI.
VS.
COMPACTIVE EFFORT
VERY SANDY
SANDY
SANDY LOAM
LOAM
CLAY SOIL
SHEARING STRENGTH - PSI WHEN NORMAL STRESS = 20 PSI.
COMPACTIVE EFFORT - FT. LB./CU. IN.
0
5
10
15
20
25
30

15

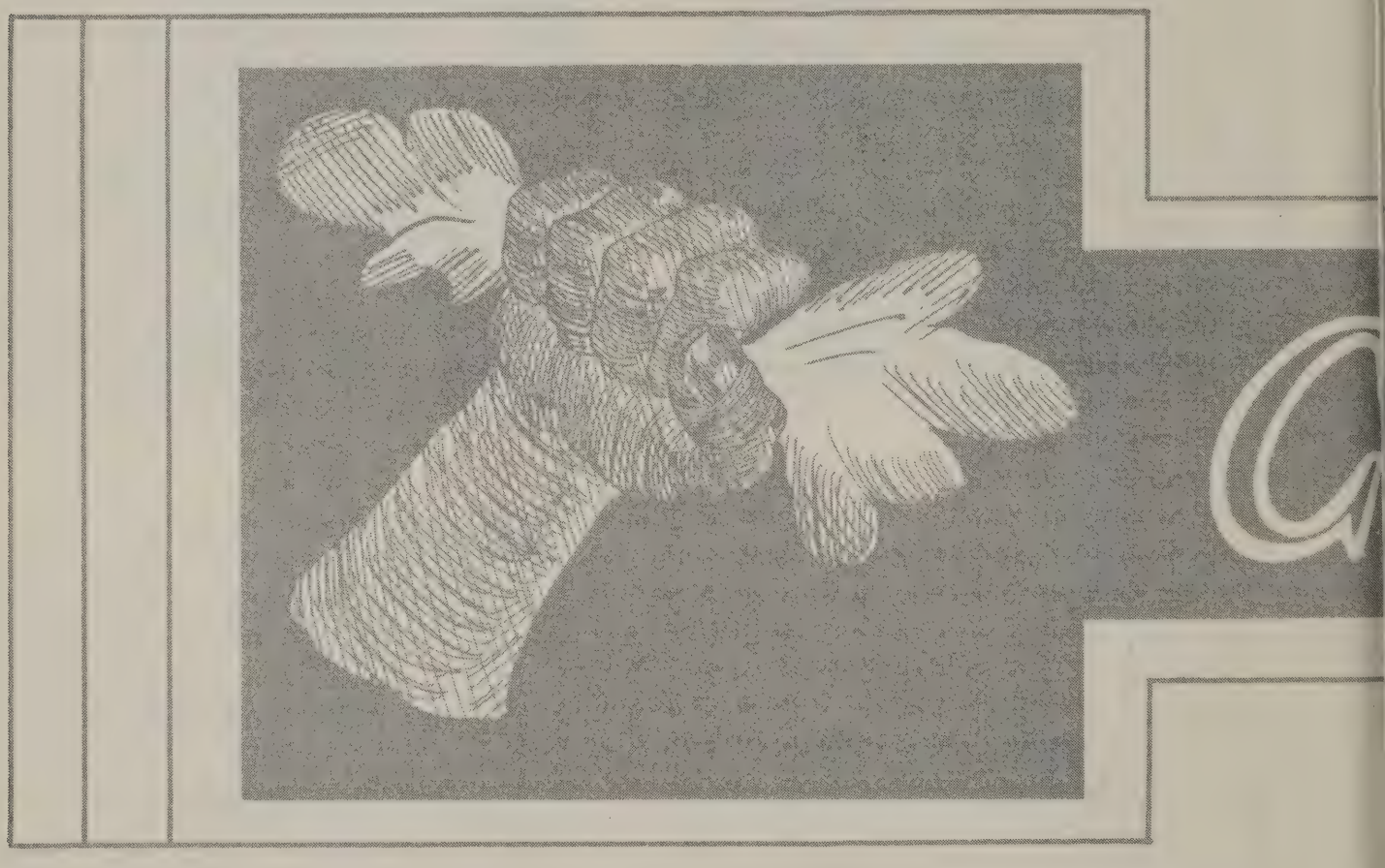
Compaction is a part of the process of converting a loose soil material into a coherent part of an engineering structure. Earthwork structures have been built since the beginning of civilization, but their control has been more of an art than a science. This art has been well known to one or more ancient civilizations and forgotten by those who followed.

The development of the science of control was spurred by the publication of the articles on the control of compaction in earth dam construction by Mr. R. R. Proctor in Engineering News Record in 1933. In this article, Mr. Proctor described a Moisture-Density test procedure, using a blow of five and one-half-pound ram stroked through twelve inches 25 times on each of three layers, in a four-inch diameter, 1/30 cubic foot mold. (Note "stroked blow.")

The shape of the Moisture-Density Curve (Figure 1) is made up of three parts: the hydration, lubrication, and the saturation legs. As pointed out by Mr. John McRea in his hypothesis, the physical character of the structure is different for material placed in each of the three conditions and continues to be different after drying and rewetting.

The A. A. S. H. O. and A. S. T. M. Committees on Soils for Engineering Purposes picked up the work of Mr. Proctor and, changing the ram blow to a free-falling drop, wrote the A. A. S. H. O. -A. S. T. M. Moisture-Density Relation Test.

The Corps of Engineers of the U. S. Army modified the A. A. S. H. O. Moisture-Density Test by changing the weight of ram to ten pounds, the height of fall to eighteen inches, the thickness of layer to one inch, and the number of layers to five, thereby increasing the compactive effort to four and one-half times the original.



The A. A. S. H. O. -A. S. T. M. Test uses 6.56 ft. lbs./cu. in. delivered by a five and one-half-pound ram dropped twelve inches on each 1 2/3-inch layer. The modified test uses 29.84 ft. lbs./cu. in. delivered by a ten-pound ram dropped eighteen inches on each one-inch layer. Both tests were limited in the 1/30 cubic foot mold to the minus 1/4 inch material.

In the meantime, California had been using a procedure employing direct compression of soil materials into a six-inch diameter mold. The Corp of Engineers adopted the six-inch

diameter mold, but used the dynamic compaction method described above. They limited particle size to 3/4 inch, replacing oversize material with an equal weight of material passing the 3/4 inch but retained on the 1/4-inch screens. This alteration of the material leaves something to be desired in that it changes the characteristics of the aggregate fraction.

The following elements make up the Moisture-Density Test:

The weight of ram and the height of fall combine to produce the force of the individual blow.

It is highly desirable to test the total material in the gradation that is to be used in construction.

The thickness of layer must be related to the compactive blow in such a way that the blow is effective throughout the layer. Also, each layer must be thick enough to contain the maximum-size particle.

The material must be prepared in such a manner as to permit the making of test specimens that duplicate the grading of the material.

The water must be mixed with the material so that all sizes get a normal wetting.

The base of the compaction ram must cover the compacted surface completely an equal number of times.

Combining these elemental requirements we have developed the Moisture-Density Test, T. H. D. -83, using the ten-pound ram, a free fall of eighteen inches, six-inch diameter mold, the sector shaped compacting surface, and two-inch layers. This test is applicable to the range of soil-bound materials up to two inches maximum particle size which includes most of our specification base materials.

COMPACTION

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Texas Highway Department
Austin, Texas

With this test procedure we have been able to develop compaction data, as shown in Figure 2, the moisture-density curves for five different compactive efforts on the same sandy soil. In Figure 3 we plot the maximum density in lbs./cu. ft. on the arithmetic scale against the compactive effort on the log scale. Figure 4 is the same data for an aggregate material.

It is noted that over the lower range of compactive effort this plot is a straight line. In the higher compactive efforts, the curve approaches solid weight asymptotically. This plot has been found very useful in determining the compactive effort required to produce a desired density.

The density of the minus 1/4-inch fraction often is a specification requirement. The limitations of the use of the density in the minus 1/4-inch fraction have caused considerable trouble in crushed rock and other materials of typical high voidage in the aggregate fraction. In Figure 5 we have diagrammed the condition that results from trying to control the compaction by density of the minus 1/4 fraction. When the amount of fines is insufficient to fill the voids of the aggregate at the density specified, this result is inevitable. It may be impossible to reduce the voids in the aggregate portion to a volume that will be completely filled by densified minus 1/4-inch material.

Mr. R. R. Proctor has established the fact that the compactive effort used in the laboratory is related to the compactive effort required to secure similar compaction in construction. He showed that the compactive effort in the field is defined by the draw ball pull on the roller, per unit volume of soil layer, per pass of the roller. To keep the rolling fully effective, the proper compactive effort must be maintained by adjusting the weight of the roller to the thickness of layer or vice versa.

The specification writers have had a tendency to require something less than 100 per cent of the maximum dry density of any test procedure, using 90, 95, and 98 per cent of the maximum density as minimum requirement. Our co-worker, Mr. Chester McDowell, has published a paper on a correlation of the desirable density to which soil materials should be compacted with the dry-rodded density of the material. In Figure 6, the upper end of the scale is "Dense Density" (the maximum density of the 30 ft. lb./cu. in. compaction) and the lower end is dry-rodded density. From his studies the attainable and desirable densities can be calculated for each material through the use of a term "compaction ratio." The required "compaction ratio" is based on the dry-rodded density. This method has been described in the T. H. D. 110. The accumulation of additional correlation data by field engineers will greatly enhance the value of this procedure.

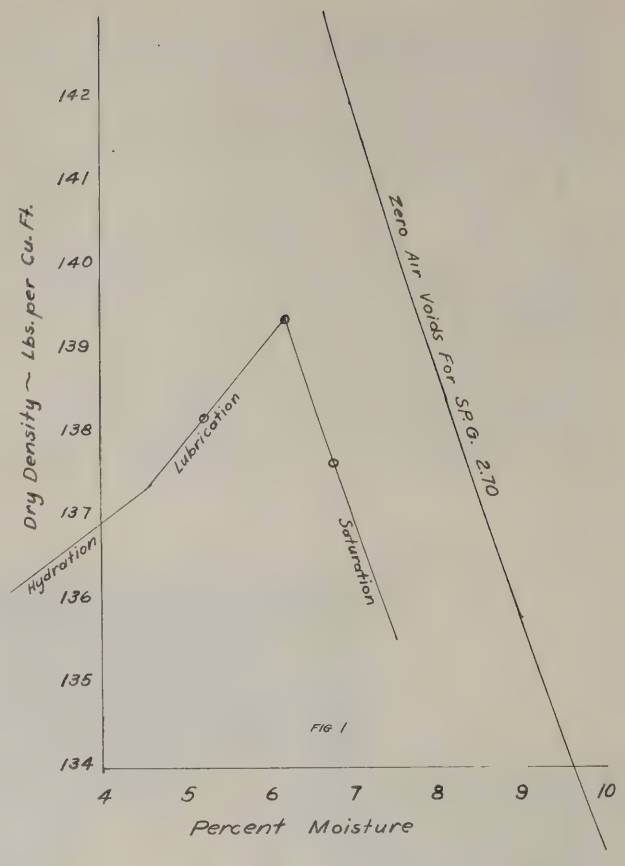


FIG 1

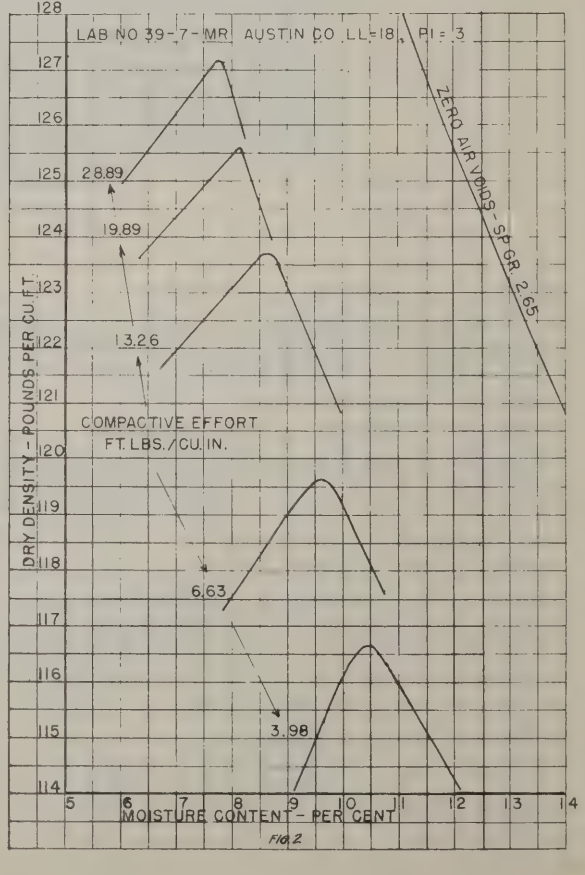


FIG 2

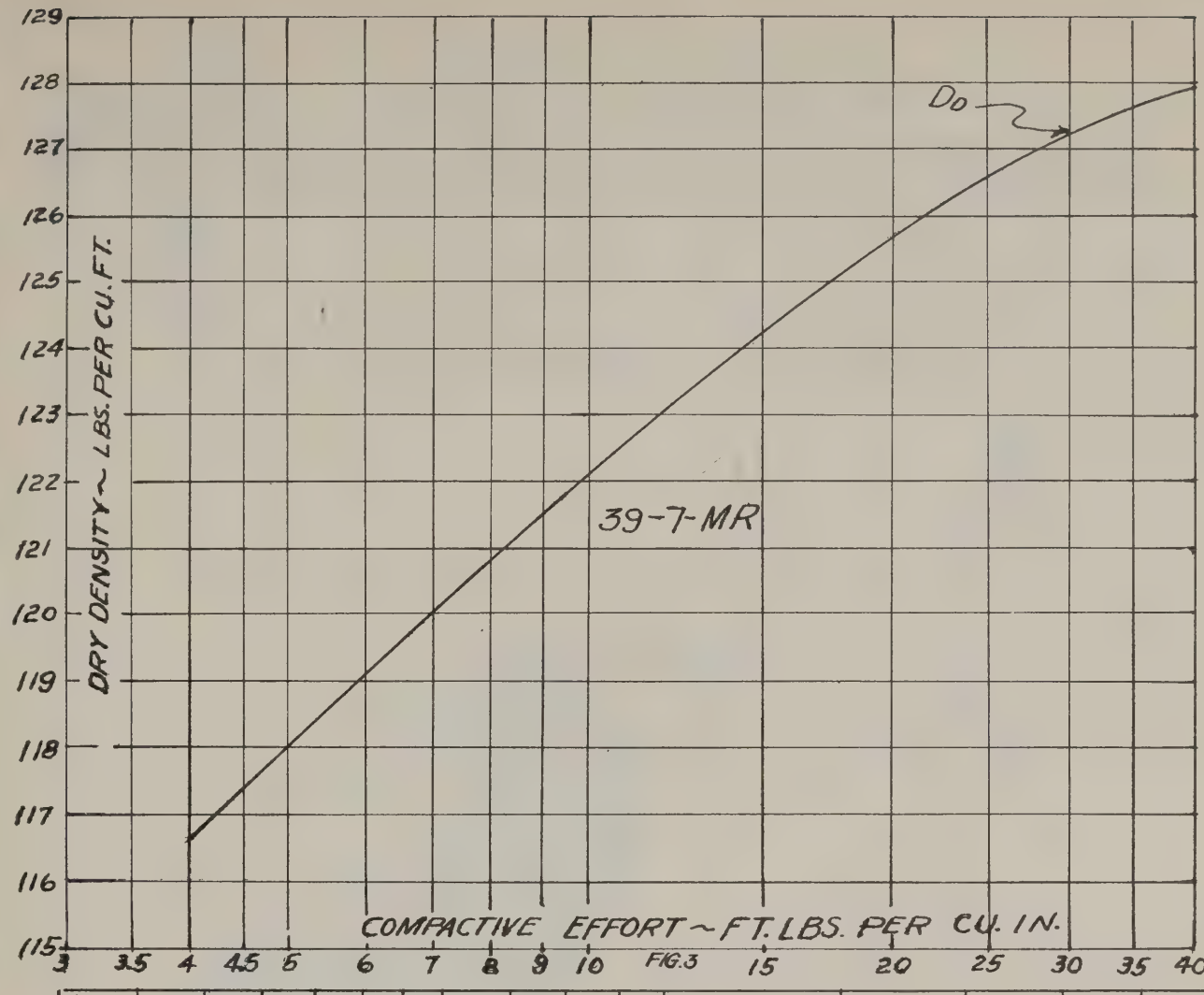

129
128
127
126
125
124
123
122
121
120
119
118
117
116
115
DRY DENSITY ~ LBS. PER CU. FT.
Do
39-7-MR
COMPACTIVE EFFORT ~ FT. LBS. PER CU. IN.
3
3.5
4
4.5
5
6
7
8
9
10
15
20
25
30
35
40

FIG. 3

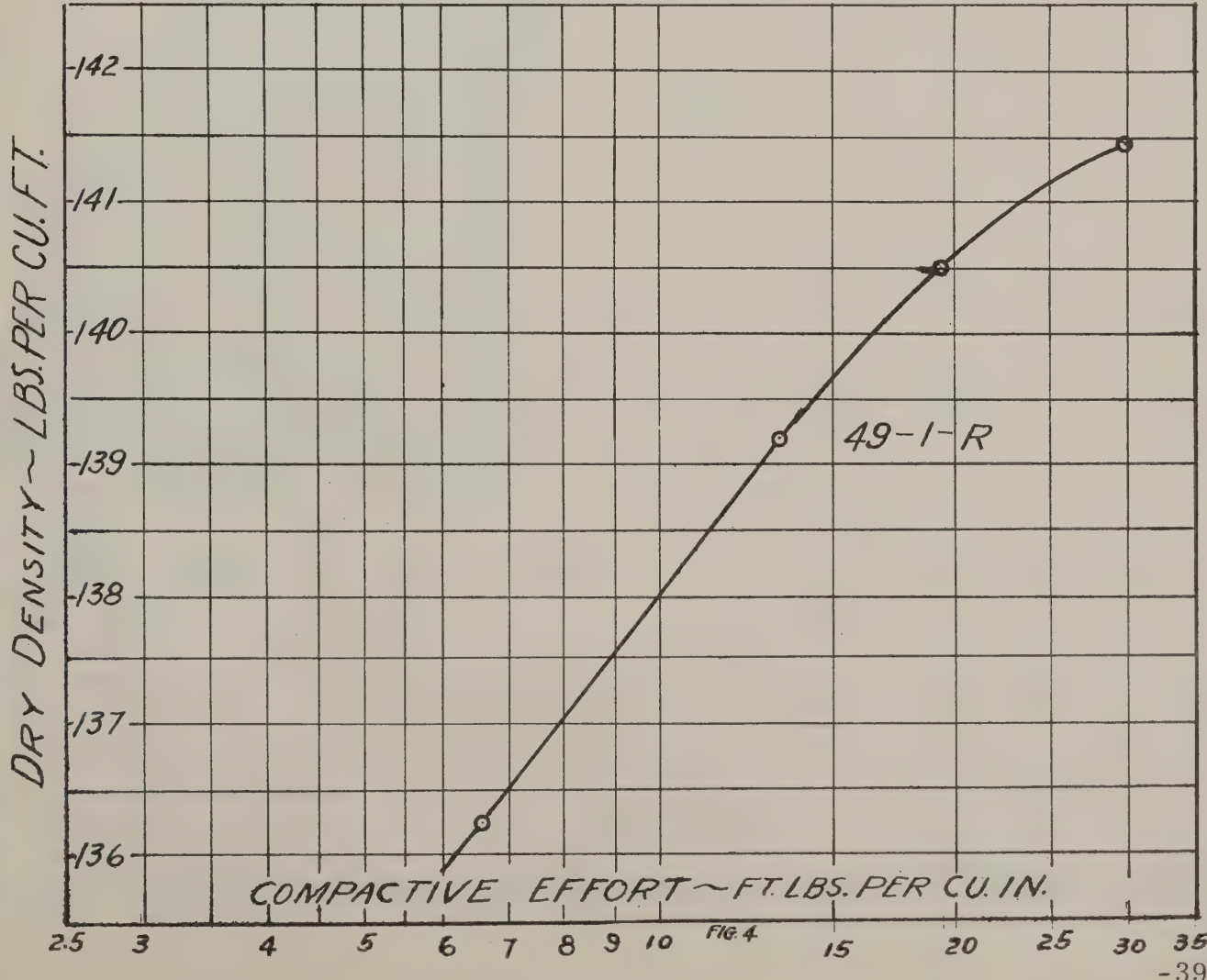
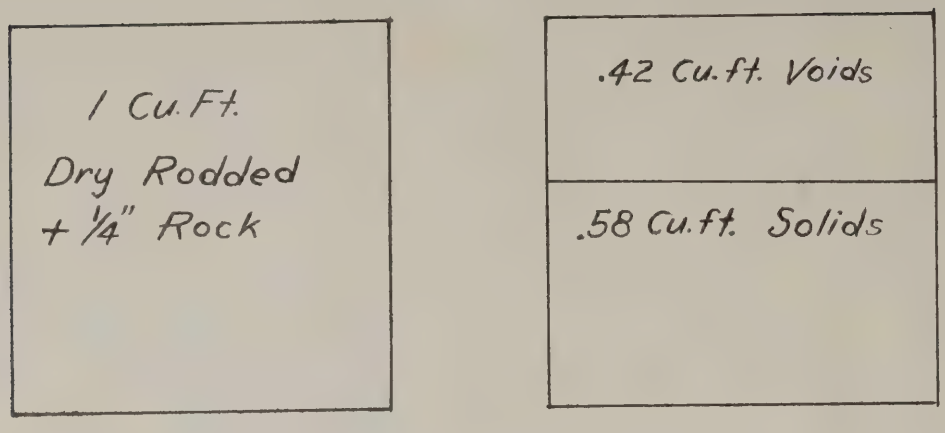

142
141
140
139
138
137
136
DRY DENSITY ~ LBS. PER CU. FT.
49-1-R
COMPACTIVE EFFORT ~ FT. LBS. PER CU. IN.
2.5
3
4
5
6
7
8
9
10
15
20
25
30
35

FIG. 4

CRUSHED ROCK

25% By Weight - ¼" & 75% + ¼"



AASHO Optimum Density of - ¼" Mat. = 115 Lbs./Cu. ft.

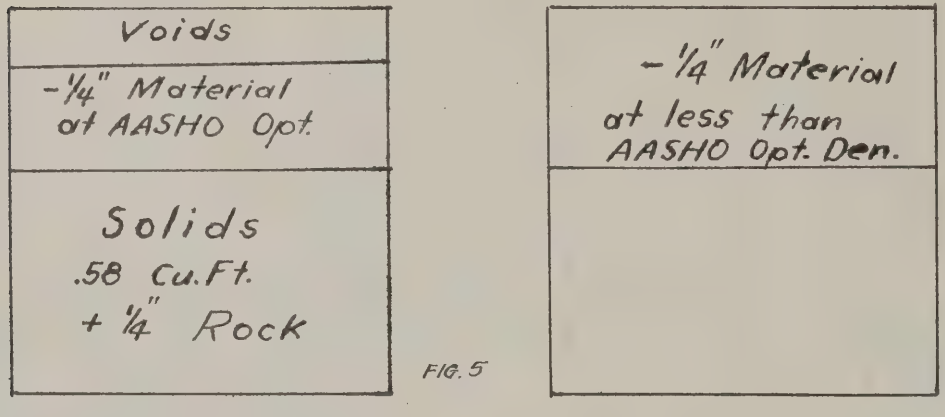


FIG. 5

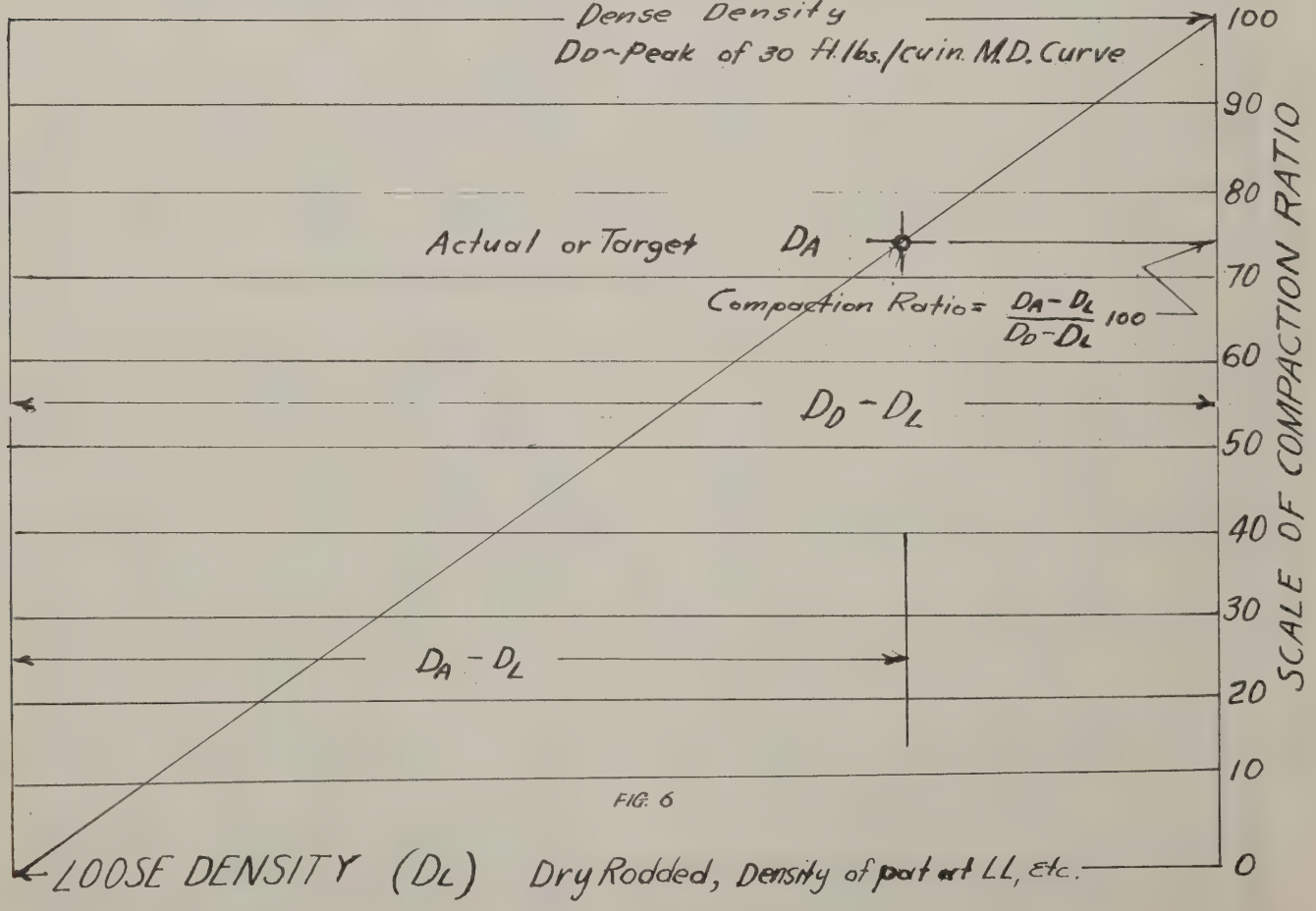
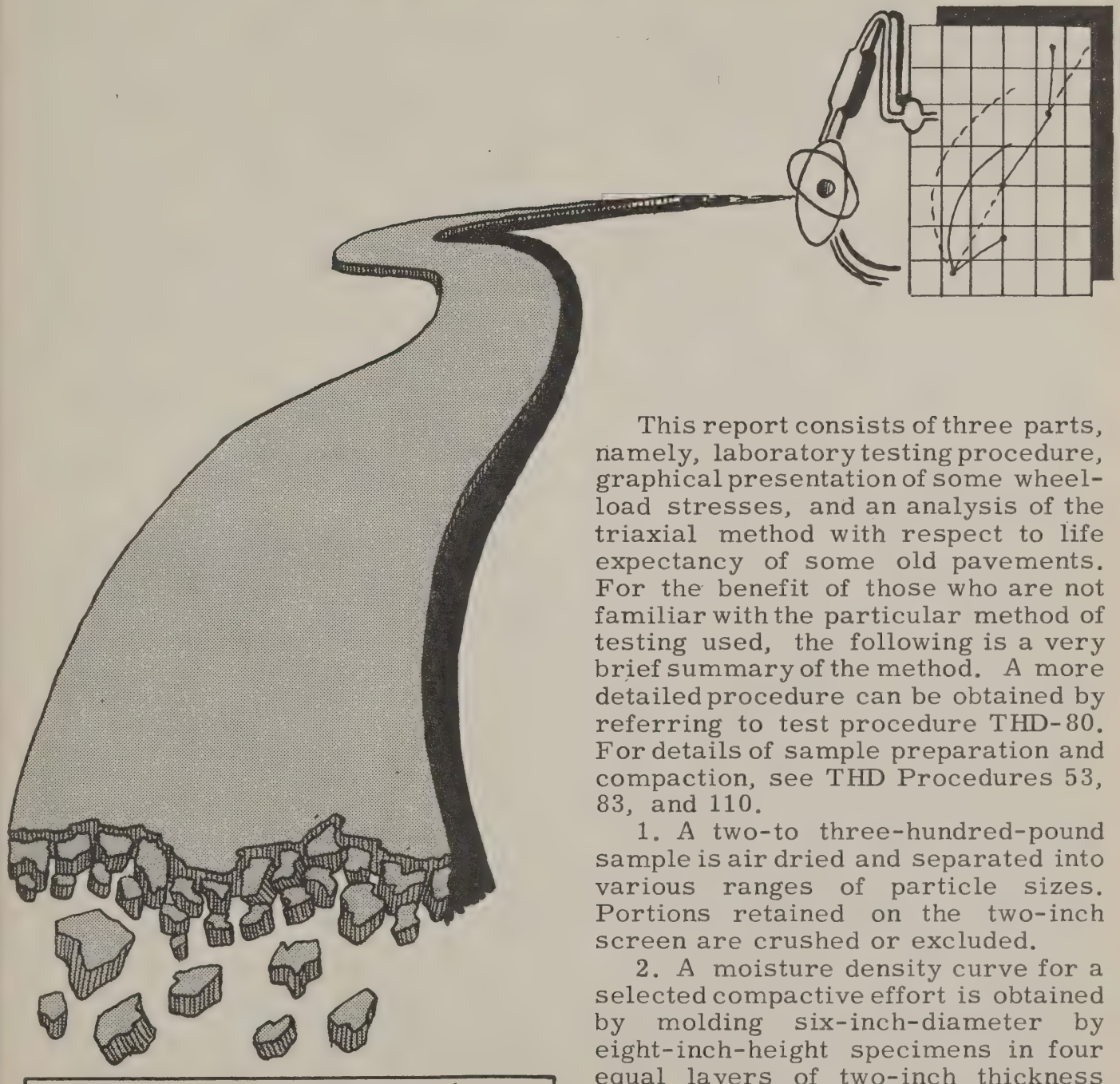


FIG. 6

Triaxial Procedure for Flexible Pavement Design



Chester Mc Dowell, Senior Soils Engineer
Texas Highway Department
Austin, Texas

This report consists of three parts, namely, laboratory testing procedure, graphical presentation of some wheel-load stresses, and an analysis of the triaxial method with respect to life expectancy of some old pavements. For the benefit of those who are not familiar with the particular method of testing used, the following is a very brief summary of the method. A more detailed procedure can be obtained by referring to test procedure THD-80. For details of sample preparation and compaction, see THD Procedures 53, 83, and 110.

1. A two-to three-hundred-pound sample is air dried and separated into various ranges of particle sizes. Portions retained on the two-inch screen are crushed or excluded.

2. A moisture density curve for a selected compactive effort is obtained by molding six-inch-diameter by eight-inch-height specimens in four equal layers of two-inch thickness each. The batch for each specimen consists of grain-size components which have been recombined on the basis of the original gradation obtained in Step 1.

3. Six specimens, as nearly identical as possible, are compacted

at optimum moisture content. All specimens are weighed, measured, and extruded from the molds. Porous stones are placed on top and bottom.

4. After storing overnight, specimens consisting of materials that do not develop shrinkage cracks are partially dried by placing in an air-drying oven (forced draught at 140°F.) for a period of eight hours. Upon removal, the specimens are allowed to stand overnight in the open laboratory. The specimens again are weighed. Usually, about two-thirds to one-half of the molding moisture is removed. Materials that tend to develop shrinkage cracks are not dried.

5. The axial cells, deflated by vacuum, are placed on the specimens. A suitable vertical surcharge (about 0.33 to 1.00 p.s.i. for most subgrade) is placed on the top stone. The specimens are then placed in pans of water so that the water level is one-half inch below the bottom of the specimen itself. This assembly is then placed on the storage rack in the moisture room and connected with the constant-pressure air manifold. The usual lateral pressure is one p.s.i. The specimens are permitted to absorb water by capillarity until equilibrium is attained.

6. Each specimen is tested in compression at a constant lateral pressure. The six identical specimens are usually tested at lateral pressures of 0, 3, 5, 10, 15, and 20 p.s.i., respectively. This pressure is applied by means of the cells, supplied by an auxiliary air tank. The rate of strain is 0.15 inch per minute. Simultaneous readings of load and deformation are taken at intervals of 0.01 inch deformation. Loading continues until the specimen fails.

7. After the completion of the compression test, the entire specimen is dried at 110°C. On the basis of the total dry weight, extra data as to density, moisture content, moisture absorption, etc., may be calculated.

8. From the principal stresses at the instant of failure, Mohr's diagram of stress is constructed.

9. A portion of the Mohr envelope is transferred to a classification chart and the strength class of the material is determined to the nearest tenth. See Figure 9, THD-80.

10. The depth of coverage in inches for a given wheel load is obtained by entering these data on a design chart. See Figure 9.

The entire procedure, when reduced to its simplest terms, consists of comparing strength to stress for all layers of flexible pavement. The strength of the material is determined from carefully controlled triaxial tests and compared to a given wheel-load stress condition by plotting the shear strength envelope on the above-mentioned classification chart. This chart has been derived from experience and certain theoretical concepts. The stress conditions for a variety of wheel loads, based upon constant tire pressures, may be analyzed by use of the pavement analysis chart, Figure 9. In an attempt to create a better understanding of the relationship of strength to stress, it is necessary to present the following portion of this report which involves some stress analysis considerations.

Wheel-load Stress Computations Related to the Texas Highway Department Triaxial Method of Flexible Pavement Design: It is recognized that no exact method for calculation of stresses beneath a wheel load exists; however, it is generally conceded that stresses existing below a uniformly loaded circular area are approximately the same as those existing beneath a pneumatic tire of similar contact and pressure intensity. It has also been conceded by many that A.E.H. Love's solution of Boussinesq's equations of elasticity presents a desirable method of estimating stresses beneath a wheel load.

The Texas triaxial method of flexible pavement design was influenced a great deal by calculations involving the above-mentioned theories. Although the Texas method has been developed largely for testing materials taken from roads of known behavior, the question is often asked about what stress considerations are pertinent to the method in use. In an attempt to answer this question the

SOME GEOMETRICAL DIMENSIONS USED BY LOVE IN THE CALCULATION OF STRESSES BENEATH A UNIFORMLY LOADED CIRCULAR AREA

NOTATIONS

a = Radius of loaded area
ρ = Radial distance in cylindrical co-ordinate's
z = Depth below surface
ρ/a = Radial distance expressed in terms of radius
z/a = Depth expressed in terms of radius
p = Applied uniform stress
$\widehat{zz}$ = Stress component acting in a vertical direction upon a horizontal plane
$\widehat{\rho\rho}$ = Stress component acting in a radial direction upon a plane perpindicular to the radius
S = Stress difference $= \sigma_I - \sigma_{III}$
μ = Poisson's ratio
$\sigma_I = \frac{1}{2}(\widehat{\rho\rho} + \widehat{zz} + S)$ Major principal stress
$\sigma_{III} = \frac{1}{2}(\widehat{\rho\rho} + \widehat{zz} - S)$ Minor principal stress*

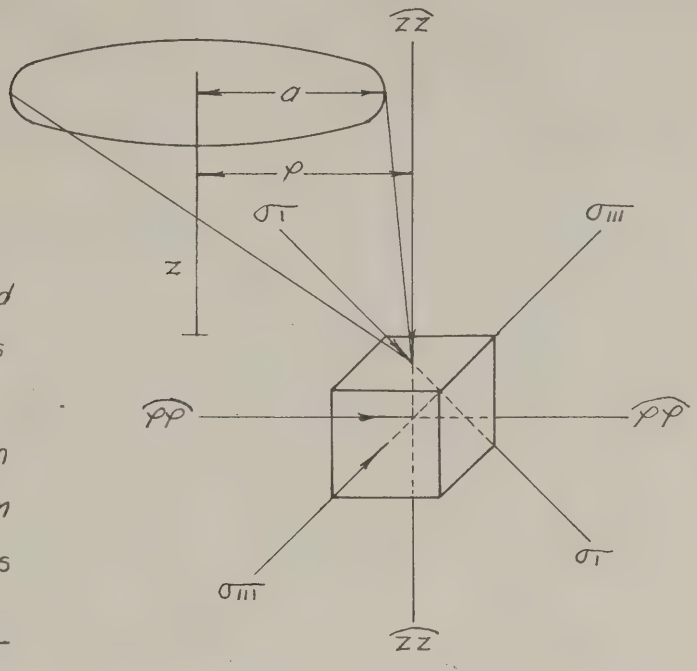


FIG. 1

*Tangential stress $\widehat{ww}$ not used. See text

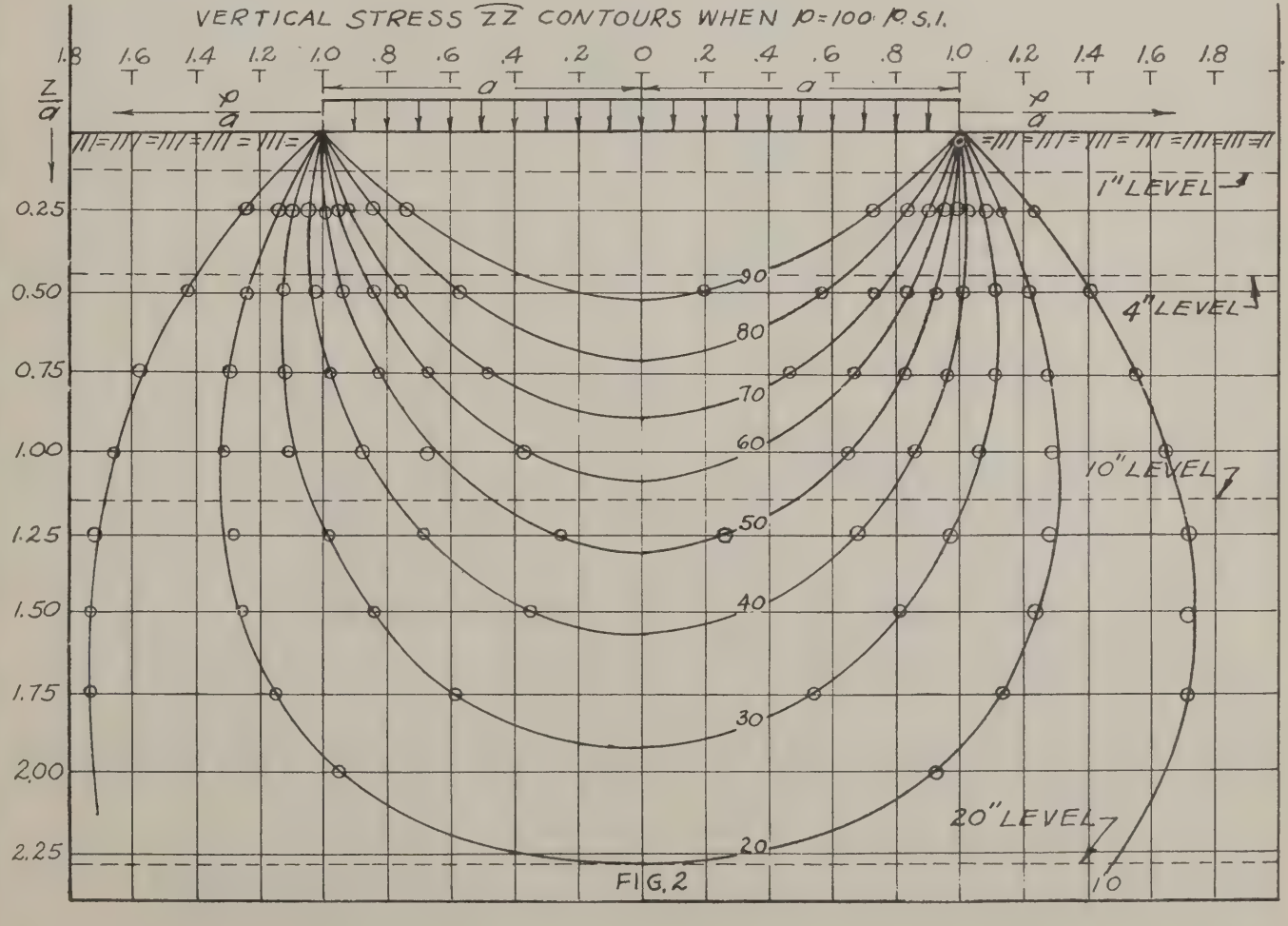
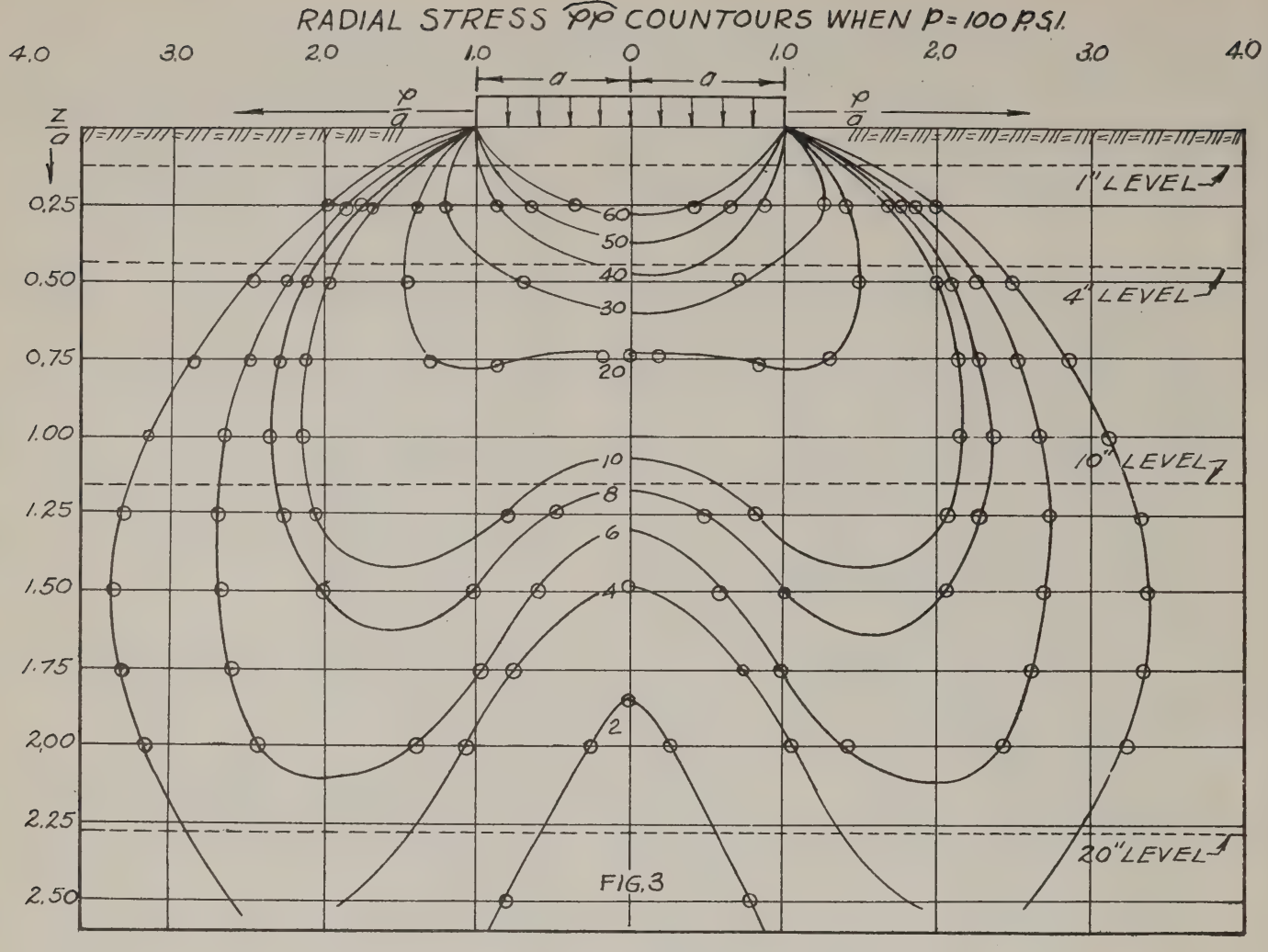
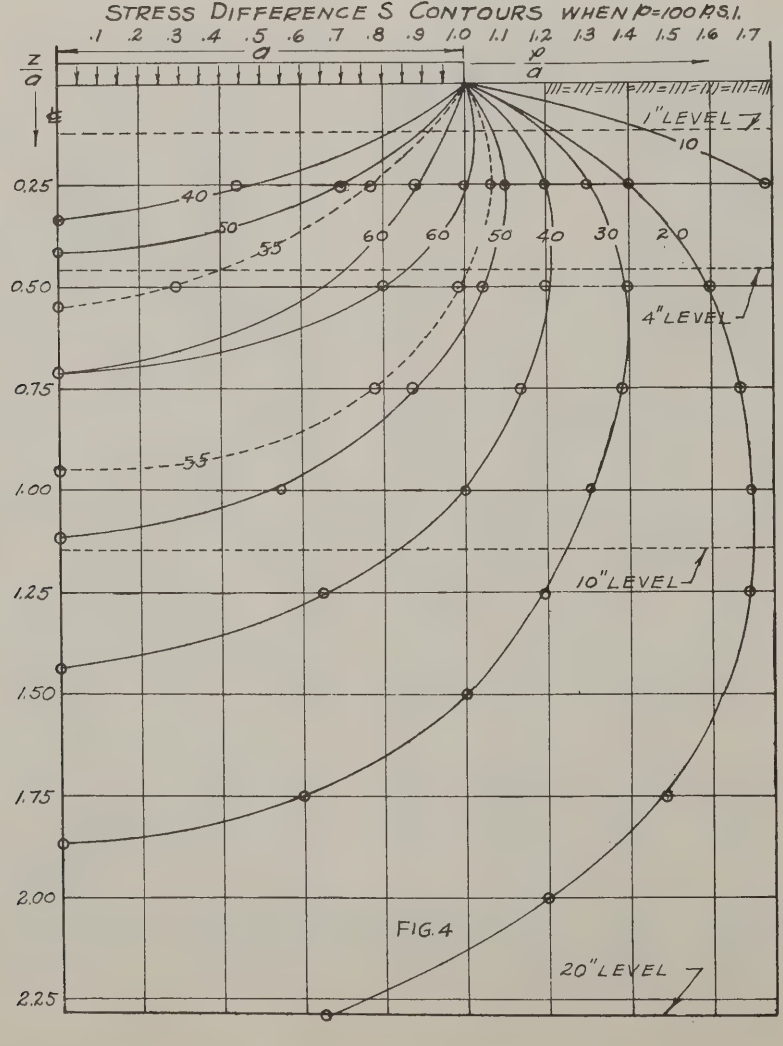


FIG. 2


RADIAL STRESS $\widehat{pp}$ COUNTOURS WHEN P=100 P.S.I.
4.0 3.0 2.0 1.0 0 1.0 2.0 3.0 4.0
a a
p/a p/a
z/a
0.25 0.50 0.75 1.00 1.25 1.50 1.75 2.00 2.25 2.50
60 50 40 30 20 10 8 6 4 2
1" LEVEL
4" LEVEL
10" LEVEL
20" LEVEL
FIG. 3


STRESS DIFFERENCE S CONTOURS WHEN P=100 P.S.I.
.1 .2 .3 .4 .5 .6 .7 .8 .9 1.0 1.1 1.2 1.3 1.4 1.5 1.6 1.7
a
p/a
z/a
0.25 0.50 0.75 1.00 1.25 1.50 1.75 2.00 2.25
40 50 55 60 60 50 40 30 20 10
55
1" LEVEL
4" LEVEL
10" LEVEL
20" LEVEL
FIG. 4

writer has tried to present some specific calculations, based on certain assumptions, which will afford a comparison between their results and those of the Texas Highway Department method. This report represents only one approach to calculation of wheel-load stresses out of many possible ones; therefore, it would not be unusual for the reader to prefer other assumptions and solutions to the problem.

ASSUMPTIONS

1. The theory of elasticity involving:

(a) A. E. H. Love's solution for stresses beneath a circular loaded area.

(b) Warner H. Tuft's computation of influence values for Love's equations.

(c) That the use of tangential stress omega-omega, $\widehat{ww}$, in a few cases where it is less than 1/2 ($\widehat{zz} + \widehat{pp} - s$) would not alter results of calculations for this problem sufficiently to justify their inclusion in this report.

(d) Reduction of shearing stresses obtained from Love's solutions due to layers of pavements having higher moduli than subgrades. This is accomplished by multiplying Love's maximum shearing stresses by a factor F_S. F_S is the ratio of the maximum shear on the axis of the loaded area from F. H. Scrivner's solution [1] for "layer two" of the two-layered system to the maximum shearing stress on the axis of loaded area from Love's solution. Mr. L. E. McCarty of the Texas Highway Department developed F_S as shown in Table II and Figure 4A. It may be noted that F_S is a function of depth Z/a and ratio of moduli E_2/E_1. A set of empirical values of E_2/E_1 based upon experience were selected by the author and are shown by dashed line in Figure 4A.

(e) The addition of effects of surcharge weight equal to one p. s. i. per foot of depth upon normal stresses for depths greater than twelve inches.

2. Poisson's Ratio $\mu = 0.5$.

3. A unit load of 100 p. s. i. With exception of pavements located in zones where traffic makes extensive use of brakes for stopping, this unit pressure is thought to be high enough to eliminate the necessity of considering effects of added stresses due to acceleration or deceleration.

4. Wheel loads of 24,000 and 10,000 pounds were used in computations. Radii of loaded areas were 8.75 and 5.6 inches, respectively.

CALCULATION DATA

1. Figure 1 shows geometrical dimensions used by Love in solution of the stress problem.

2. Table I lists Warner H. Tuft's computation of influence values for Love's equations.

3. Figures 2, 3, and 4 show the distribution of vertical stresses, radial or horizontal stresses and stress differences under a uniformly loaded circular area. The data for plotting of these figures were taken from Warner Tuft's tables of influence values, assuming an applied unit load of 100 p. s. i.

4. Figure 5 shows plottings of Mohr diagram of stress for the one-inch level beneath a circular loaded area having a unit pressure of 100 p. s. i. and a radius 8.75 inches. The stress value of $\widehat{zz}$ for vertical, $\widehat{pp}$ for radial, and $S/2$ for maximum shear which were used in plotting of the Mohr diagrams were interpolated from Figures 2, 3, and 4. These values are tabulated in the upper right part of the Mohr diagram chart. Similar plottings for 4-, 10-, and 20-inch levels were also made but are not shown in this report.

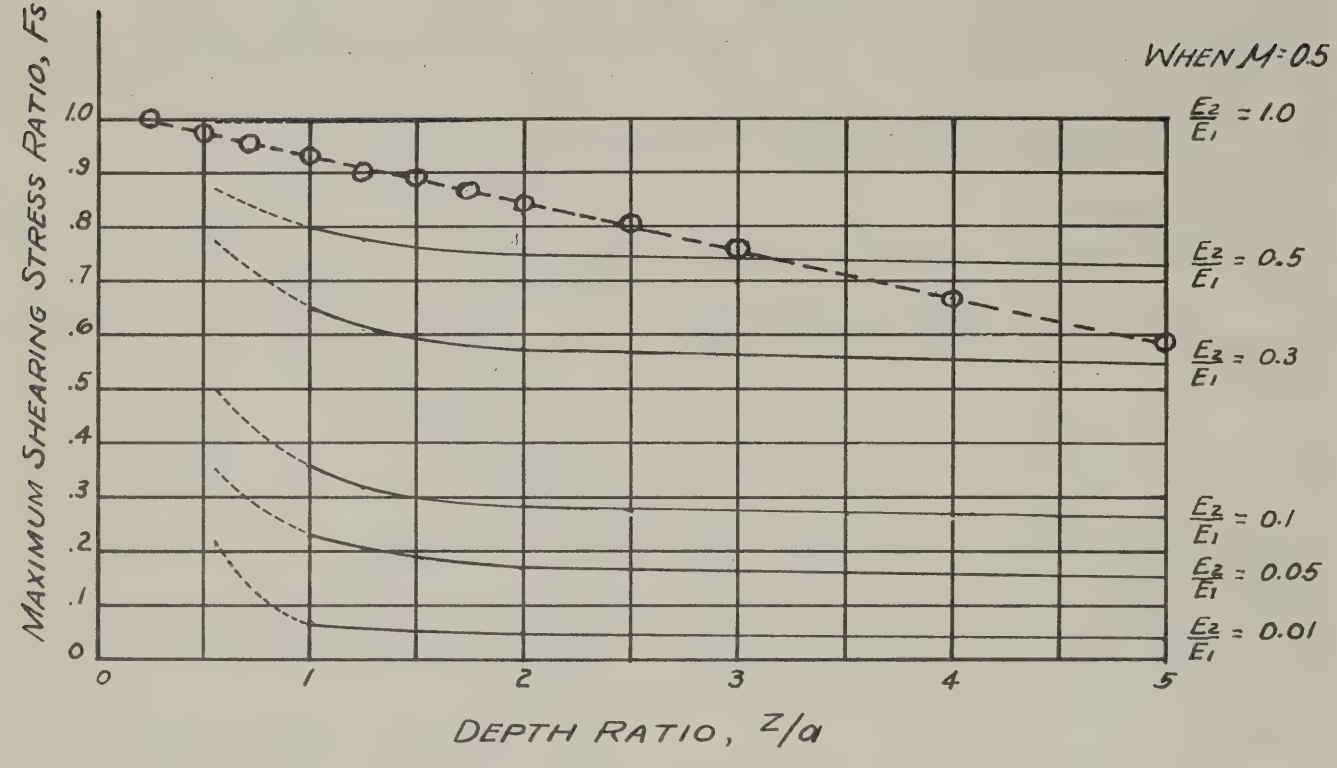
5. Figure 6 shows the stress envelopes superimposed on the Texas triaxial classification chart. The lowest (lowest graphically meaning weakest class) classes of soil materials strong enough to pass above the calculated stress envelopes are shown on left side of chart.

The foregoing method of presenting stresses, i. e., by the use of Figures 2, 3, and 4, is far from the easiest but is used in this instance to present a picture of stress distribution. For those who are accustomed to using

[1] Some Numerical Solutions of Stresses in Two and Three Layered Systems, by F. H. Scrivner, Volume 28, Proceedings of the Highway Research Board.

SHEARING STRESS CORRECTION RATIO, Fs, FOR CONVERTING LOVES INFLUENCE STRESSES TO THOSE FOR A TWO LAYERED SYSTEM

FIG. 4-A



ONE INCH LEVEL P=100 P.S.I., μ=0.5, a=8.75" F_w = 1.00

$\overline{zz}$	$\overline{xy}/a$	$\overline{xx}$	z/a	center $\overline{zz}/2+\overline{xx}/2$	Radius s/2
96	℄	85	0.114	90	8
90	.88	50	"	70	25
80	.94	46	"	63	30
70	.96	44	"	57	31
60	.98	42	"	51	31
50	1.00	40	"	45	31
40	1.02	38	"	39	30
30	1.04	38	"	34	30
20	1.06	36	"	28	28
10	1.12	34	"	22	21

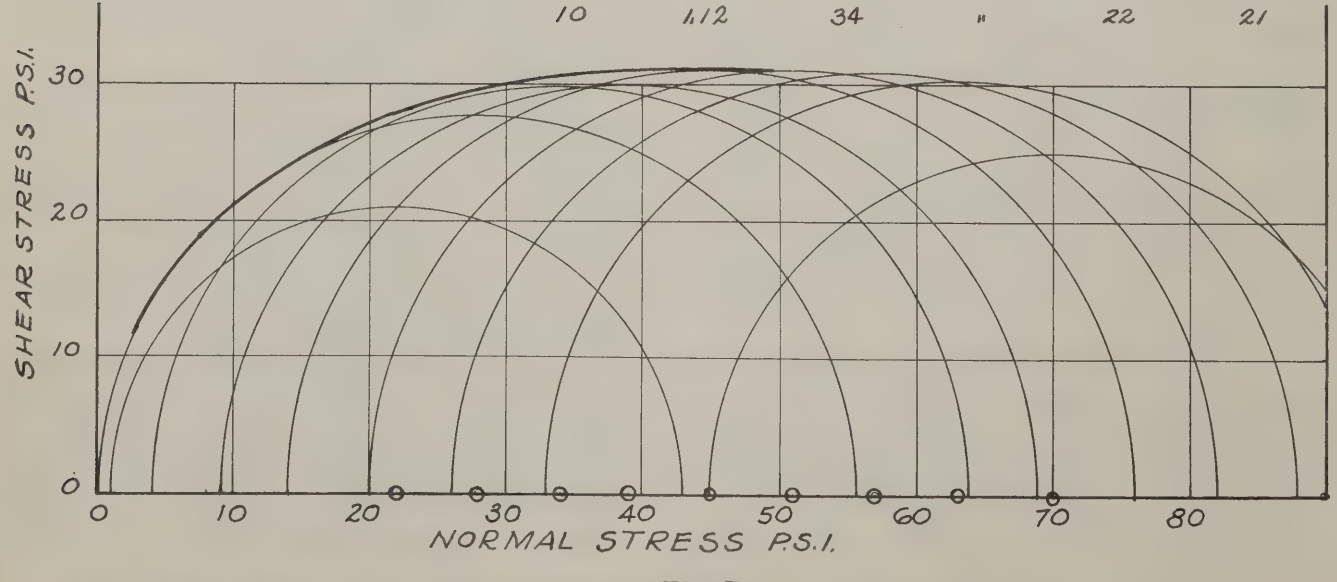


FIG. 5

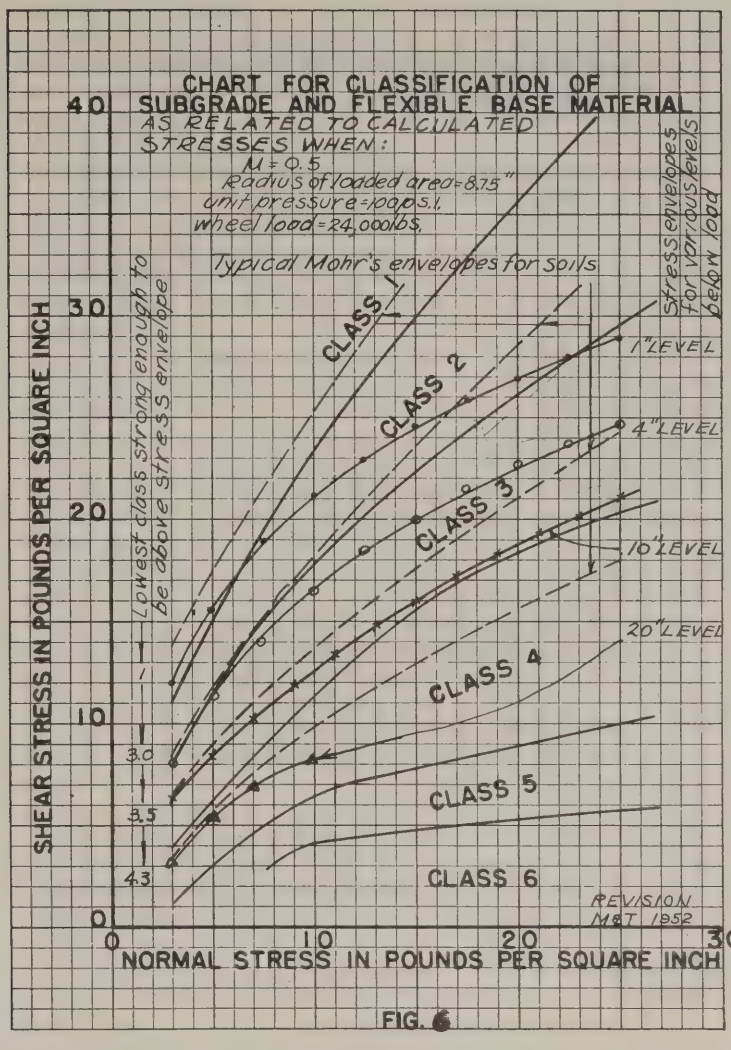


FIG. 6

14 INCH LEVEL, P = 100 P.S.I., M = 0.5, a = 5.6", F_S = 0.81

$\widehat{ZZ}$	P/a	$\widehat{PP}$	Z/a	Center $\frac{\widehat{ZZ}+\widehat{PP}}{2}$	S/2	Radius S/2 × 0.81
20	0	0.8	2.5	10.4	9.6	7.8
16.8	0.7813	1.8	"	9.3	8.7	7.0
13.3	1.2187	2.8	"	8.0	7.6	6.2
6.2	2.1658	4.0	"	5.1	5.0	4.1
3.6	2.7505	3.8	"	3.7	3.6	2.9

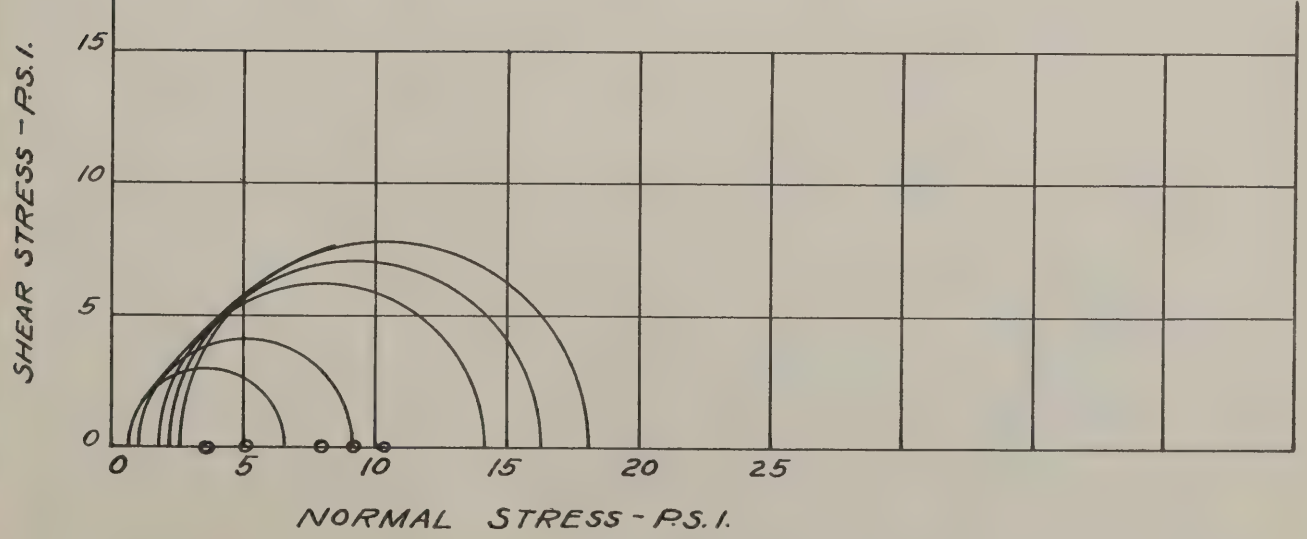


FIG. 7

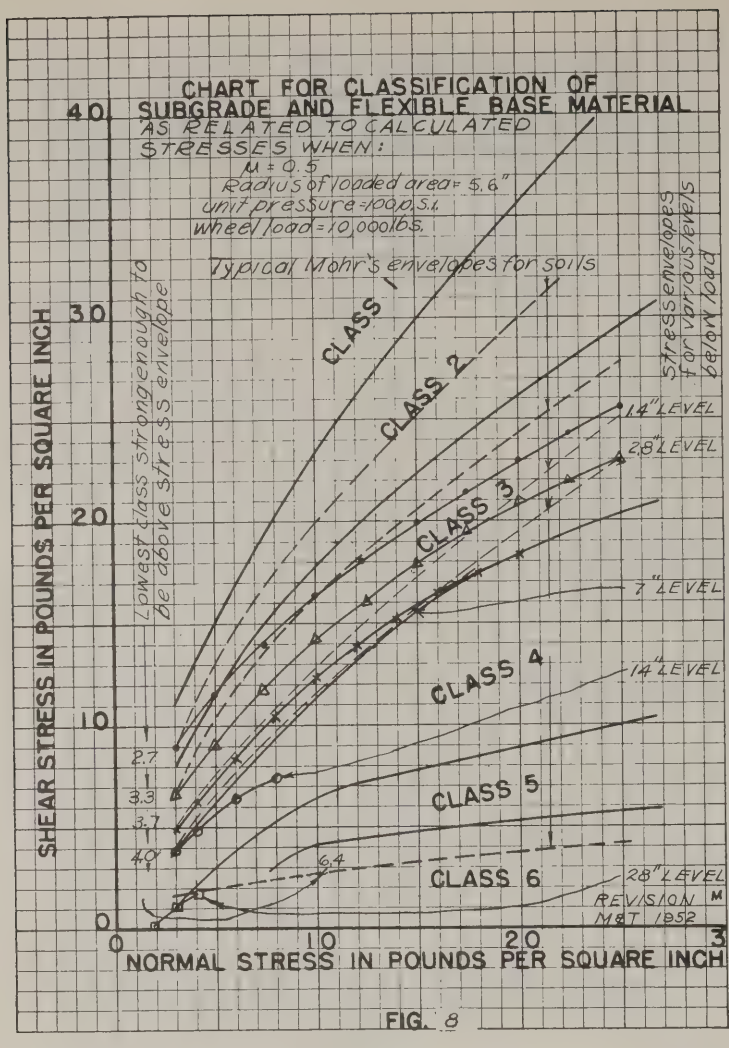


CHART FOR CLASSIFICATION OF SUBGRADE AND FLEXIBLE BASE MATERIAL
AS RELATED TO CALCULATED STRESSES WHEN:
M = 0.5
Radius of loaded area = 5.6"
unit pressure = 100 p.s.i.
wheel load = 10,000 lbs.
Typical Mohr's envelopes for soils
Lowest class strength enough to be above stress envelope
Stress envelopes for various levels below load
SHEAR STRESS IN POUNDS PER SQUARE INCH
NORMAL STRESS IN POUNDS PER SQUARE INCH
CLASS 1
CLASS 2
CLASS 3
CLASS 4
CLASS 5
CLASS 6
14" LEVEL
28" LEVEL
7" LEVEL
14" LEVEL
28" LEVEL
2.7
3.3
3.7
4.0
6.4
REVISION M&T 1952
0
10
20
30
40

FIG. 8

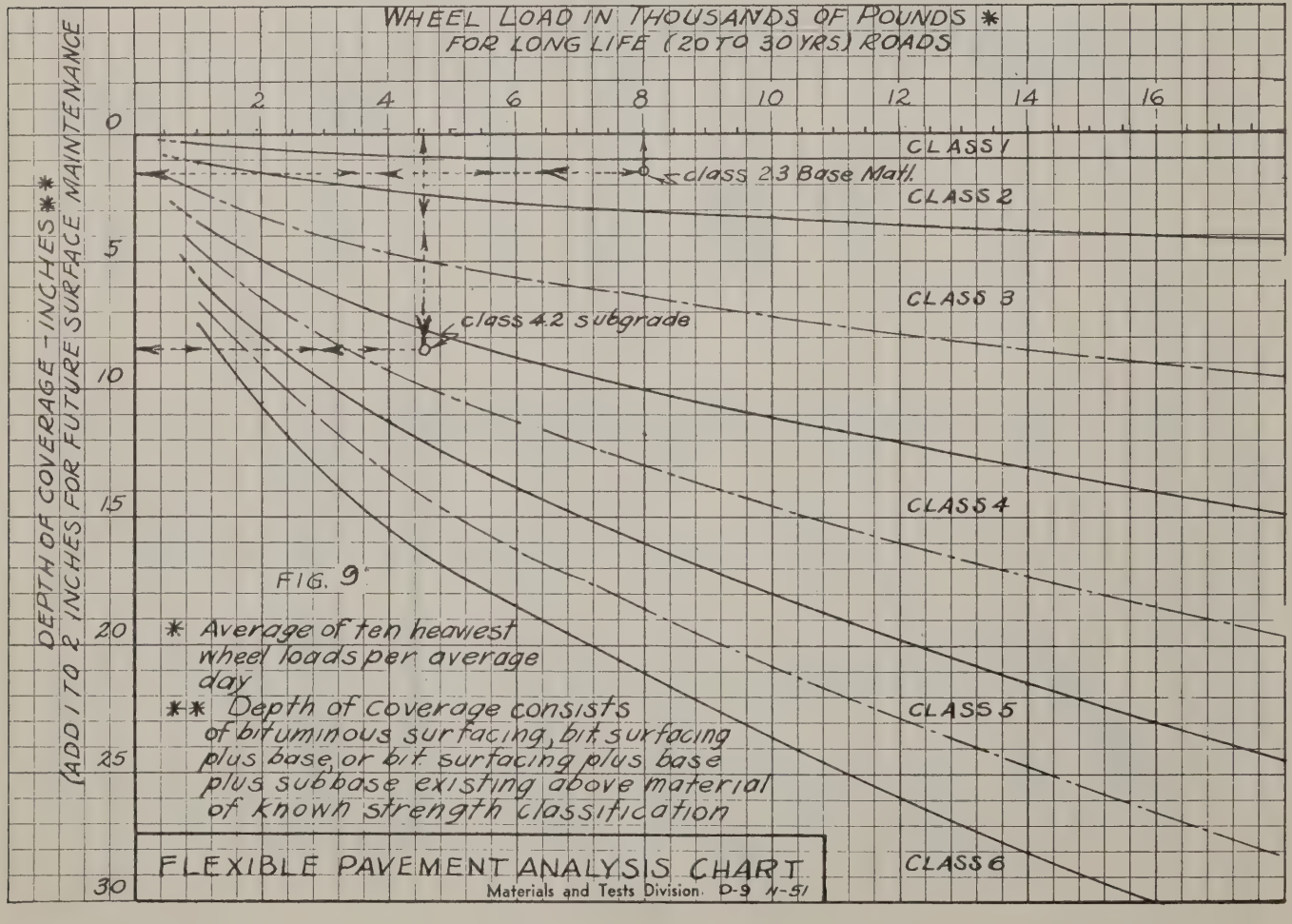

WHEEL LOAD IN THOUSANDS OF POUNDS *
FOR LONG LIFE (20 TO 30 YRS.) ROADS
DEPTH OF COVERAGE - INCHES **
(ADD 1 TO 2 INCHES FOR FUTURE SURFACE MAINTENANCE
2
4
6
8
10
12
14
16
0
5
10
15
20
25
30
CLASS 1
class 2,3 Base Matl.
CLASS 2
CLASS 3
class 4,5,6 subgrade
CLASS 4
CLASS 5
CLASS 6
* Average of ten heaviest wheel loads per average day
** Depth of coverage consists of bituminous surfacing, bit. surfacing plus base, or bit. surfacing plus base plus subbase existing above material of known strength classification
FLEXIBLE PAVEMENT ANALYSIS CHART
Materials and Tests Division D-9 4-51

FIG. 9

tables of influence values and who are not concerned with values for an exact depth will find it easier to plot directly from tables of influence values.

6. Figure 7 shows a plotting of the Mohr diagram of stress where values for vertical, radial, and maximum shearing stresses were taken directly from Mr. Tuft's tables for a depth of fourteen inches beneath a circular loaded area having a unit load of 100 p.s.i. and a radius of 5.6 inches. Similar plottings for 1.4-, 2.8-, 7-, and 28-inch levels were also made, but are not shown in this report. The depth levels represent Z/a ratios of 0.25, 0.50, 1.25, 2.50, and 5.00, respectively, and influence values for stresses at such depths can be taken directly from tables. These data are used for plotting of the Mohr diagrams and are shown in the upper right-hand portion of Figure 7.

7. Figure 8 shows stress envelopes superimposed on the Texas classification chart. The lowest class of soil strong enough to pass above calculated stress envelopes are also shown.

8. A tabulation was prepared showing depth levels at which stress computations were made, the weakest triaxial classifications are subject to overstressing, and the depths required for long-life roads built on such soil materials. These depths are taken from the Texas wheel-load analysis chart, Figure 9. This chart has been worked out on the following basis:

(a) Depths are known for a given wheel load.

(b) Tire pressures remain constant when wheel loads vary and the magnitude of radii of contact areas will change accordingly.

Then: Depth for desired design wheel load =

$$\text{Known depth for given wheel load} \sqrt{\frac{\text{wheel load desired for design}}{\text{wheel load for which depths are known}}}$$

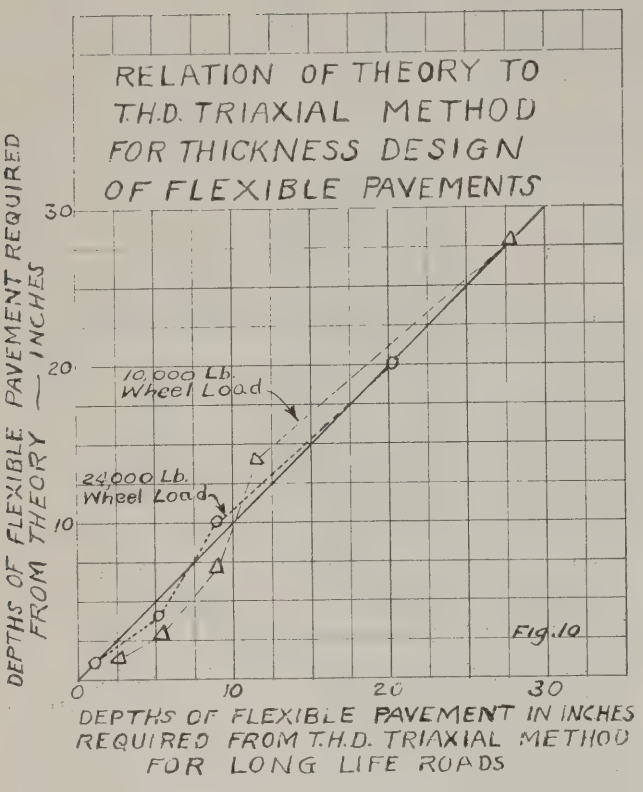
Data obtained in the above mentioned tabulation were used to make a comparison of depths shown in Figure 10 which indicates that the Texas triaxial method requires approximately the same depths as those required by the theoretical concepts presented herewith.

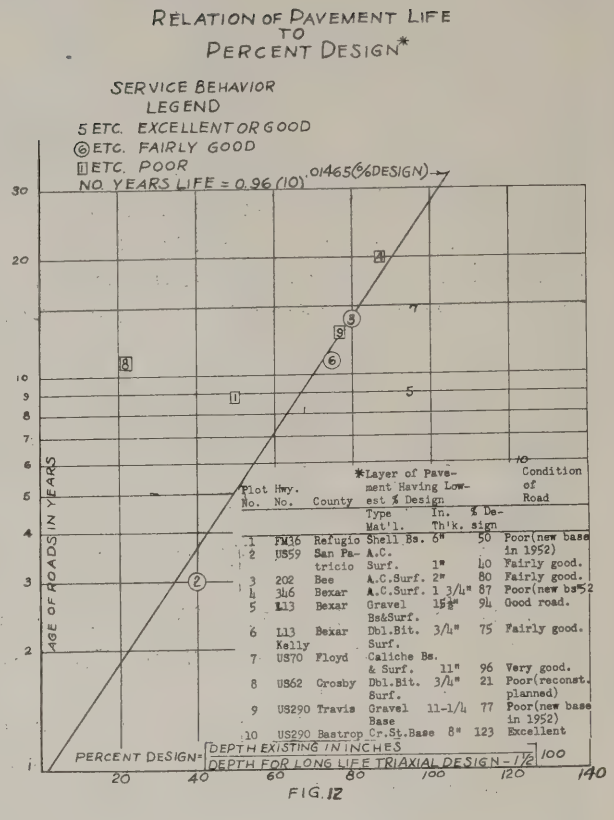
The foregoing shows that a correlation exists between a given laboratory test procedure, a given design procedure, and a certain stress analysis approach, but in the final analysis the true evaluation of the methods must be based upon tests obtained from materials used on roads of known behavior. Therefore, the third and final portion of this report is presented for your consideration.

Triaxial Tests Used in the Analysis of Existing Roads: The primary purpose of this portion of the report is to show how well this method of analysis correlates with actual service behavior of flexible pavements. In order to do this, a thorough study of pavements, including service behavior, age, weight, and volume of traffic, thicknesses of pavement layers, triaxial tests, soil constants and gradation on subgrades, subbases and bases, was made on ten projects located in South, Central, and West Texas. Samples weighing two to three hundred pounds each were taken from each layer of subgrade, subbase, or base after cutting large holes in existing pavements. At this time thicknesses of all layers of pavements were measured. The age of the pavement and an evaluation of its service behavior were obtained from State Highway District and Bureau of Public Roads personnel. The above-mentioned tests were performed and results are shown on form sheets 476-A.

In order to study these projects objectively, estimates of the average of the ten heaviest wheel loads per average day during the life of these pavements were obtained from the Texas Highway Department Planning Survey Division. Since these are averages over a period of years, it may be noted that many of the old projects showed much lower wheel-load averages than are found on newer projects.

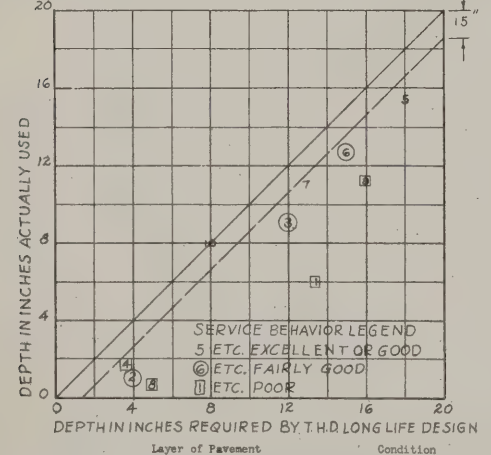
Wheel-load data, together with pertinent test results, are shown in the summary tabulation. Data shown in this tabulation, for layers of pavement needing greatest increase in thickness as judged by triaxial design, were plotted as shown in Figure 11, so as to show the relation of existing depths


RELATION OF THEORY TO T.H.D. TRIAXIAL METHOD FOR THICKNESS DESIGN OF FLEXIBLE PAVEMENTS
DEPTHS OF FLEXIBLE PAVEMENT REQUIRED FROM THEORY — INCHES
10,000 Lb. Wheel Load
24,000 Lb. Wheel Load
Fig.10
DEPTHS OF FLEXIBLE PAVEMENT IN INCHES REQUIRED FROM T.H.D. TRIAXIAL METHOD FOR LONG LIFE ROADS


RELATION OF PAVEMENT LIFE TO PERCENT DESIGN*
SERVICE BEHAVIOR LEGEND
5 ETC. EXCELLENT OR GOOD
⑥ ETC. FAIRLY GOOD
1 ETC. POOR
NO. YEARS LIFE = 0.96 (10)^.01465(%DESIGN)
AGE OF ROADS IN YEARS
PERCENT DESIGN = DEPTH EXISTING IN INCHES / DEPTH FOR LONG LIFE TRIAXIAL DESIGN - 1½ × 100
FIG. 12

Plot No.	Hwy. No.	County	*Layer of Pavement Having Lowest % Design: Type Mat'l.	In. Th'k.	% Design	Condition of Road
1	FM36	Refugio	Shell Bs.	6"	50	Poor(new base in 1952)
2	US59	San Patricio	A.C. Surf.	1"	40	Fairly good.
3	202	Bee	A.C.Surf.	2"	80	Fairly good.
4	346	Bexar	A.C.Surf.	1 3/4"	87	Poor(new bs'52
5	L13	Bexar	Gravel Bs&Surf.	15⅜"	94	Good road.
6	L13 Kelly	Bexar	Dbl.Bit. Surf.	3/4"	75	Fairly good.
7	US70	Floyd	Caliche Bs. & Surf.	11"	96	Very good.
8	US62	Crosby	Dbl.Bit. Surf.	3/4"	21	Poor(reconst. planned)
9	US290	Travis	Gravel Base	11-1/4	77	Poor(new base in 1952)
10	US290	Bastrop	Cr.St.Base	8"	123	Excellent

RELATION OF ACTUAL DEPTHS USED TO DEPTHS REQUIRED BY T.H.D. TRIAXIAL TEST FOR LAYERS OF PAVEMENT REQUIRING GREATEST THICKNESS OF COVERAGE


DEPTH IN INCHES ACTUALLY USED
DEPTH IN INCHES REQUIRED BY T.H.D LONG LIFE DESIGN
15"
SERVICE BEHAVIOR LEGEND
5 ETC. EXCELLENT OR GOOD
⑥ ETC. FAIRLY GOOD
1 ETC. POOR

Plot No.	Highway No.	County	Layer of Pavement Requiring Thickest Cover: Type Mat'l.	Inches Thickness: Existing	For Design	Condition of Road
1	FM-36	Refugio	Shell Base	6	13-1/2	Poor (new base in '52)
2	US-59	San Patricio	A.C. Surface	1	4	Fairly good.
3	202	Bee	Caliche Base	9	12	Fairly good.
4	346	Bexar	A.C. Surface	1-3/4	3-1/2	Poor (new base in '52)
5	Loop 13	Bexar	Gravel Base & Surface	15-1/2	18	Good road.
6	Loop 13 Kelly	Bexar	Dbl. Bit. Surf.	12-3/4	15	Fairly good.
7	US-70	Floyd	Caliche Base & Surface	11	13	Very good.
8	US-62	Crosby	Dbl. Bit.	3/4	5	Poor (reconstruction planned.)
9	US-290	Travis	Gravel	11-1/4	16	Poor (new base in '52)
10	US-290	Bastrop	Cr. St. Base	8	8	Excellent

FIG. 11

SUMMARY TABULATION

Highway No.	County	Road Age Yrs.	Wheel Load* (in Pounds)	Pavement Layer Thickness (inches)	Triaxial Strength Class	No. Inches Coverage Required by Tests**	Percent Design***	Service Behavior of Road
FM-136	Refugio	9	6,100	¾" single asph. surf. 5¼" shell	1	1	****	New base placed over old base in 1952
				Clay Subg.	4.9	13½	50	
US-59	San Patricio	3	12,500	1" A.C. surfacing 10" caliche	3.1	4	40	Fairly good.
				Subgrade	3.4	7	200	
202	Bee	14	5,000	2" A.C. Surfacing 7" caliche	3.3	4	80	Fairly good.
				Subgrade	4.8	12	86	
346	Bexar	20	4,500	1-3/4" A.C. surfacing 13" gravel	3.1	3½	87	Poor; new base placed over old base in 1952
				Subgrade	3.1	3½	737	
Loop 13 Lackland Air bs.	Bexar	9	8,200	2½" A.C. surfacing 13" gravel	2.8	2-3/4	200	Good.
				Subgrade	5.2	18	94	
Loop 13 Kelly Overpass	Bexar		8,100	3/4" dbl. bit. surf. 12" gravel	2.7	2½	75	Fairly good.
				Subgrade	4.7	15	94	
US-70	Floyd	14	6,700	2" triple bit. surf. 9" caliche	3.1	3½	100	Excellent
				Subgrade	4.6	13	96	
US-62	Crosby	11	7,900	3/4" dbl. bit. surf. 7" caliche	3.4	5	21	Reconstruction planned.
				Subgrade	4.4	12	74	
US-290	Travis	13	8,400	1-1/4" triple bit.surf. 10" gravel	3.0	3	83	Reconstructed after 13 yrs. service.
				Subgrade	4.9	16	77	
US-290	Bastrop	6	16,000	1/4" single bit. surf. 7-3/4" cr. stone base	1	1	****	Excellent
				Subgrade	3.4	8	123	

*Estimated average of 10 heaviest wheel loads per av. day during years road has been in service. Data furnished by THD Highway Planning Survey Division.
**Triaxial tests for long-life design.
***Percent Design = (Depth existing divided by depth required by tests minus 1½ in.)100
****Percent design is very high but cannot be determined from the formula.

Texas Highway Department
Form 473A 7-58-5m

84005-758-5m

SOILS AND BASE MATERIALS TEST REPORT

Laboratory No. _____
Date Rec'd _____ Reported _____
Engineer _____
Address _____
Contractor _____
Sampler H. S. Gillette & C. McDowell
Sampler's Title _____
Sampled From Roads
Producer _____
Quantity Represented by Sample _____
Has been Used on _____

Control Number | Section Number | Job Number
County 16 | Federal Project No. | Highway No.
District No. | L.P.E. No. | Req. No. | Date Sampled
Specification Item No. _____
Material from Property of _____
Proposed for Use as _____

Lab. No.	LL	PI	FME	CME	SL	LS	SR	Class	Soil Binder	% Moist.	Str. Class
50-76-R	32	18	23	-	19	6.8	1.74	A-6-2	26	- --	1.0
77	40	23	25	-	16	11.5	1.86	A-6	100	14.8	4.9
72	38	15	30	-	25	6.0	1.61	A-4-7	49	--	3.1
73	30	15	27	-	17	6.6	1.80	A-2	89	9.4	3.4
74	49	22	34	-	28	8.9	1.52	A-7	45	--	3.3
75	56	35	28	-	14	17.6	1.90	A-7	98	18.1	4.8

PERCENT RETAINED ON

Lab. No.	Round Opening Screens: Opening in Inches 2½	2	1½	1	¾	½	¼	Square Mesh Screens: Sieve Numbers 10	20	40	60	100	200	Grain Diam. in Millimeters .05	.005	.001	Specific Gravity
50-76-R				4	11	16	25	44	59	71	74	75	77	88	89	93	97	2.64
77							1	3	32	45	73	87						2.65
72				2	4	8	12	22	33	43	51	56	61	67	70	90	98	2.64
73							1	2	3	11	25	40	65	73	90	96		2.60
74				1	4	9	15	27	42	51	55	59	62	67	70	93	97	2.65
75										0	2	9	25	51	56	66	83	2.66

SAMPLE IDENTIFICATION

Lab. No.	Identification Marks	Location—Properties—Station Numbers	Type of Materials
50-76-R	No, 1, FM-136	Refugio Co. (between Bayside and Woodsboro)	5½" of shell base Subgrade soil
77	" "		
72	No. 3 US 59	Business route in Mathis, 3 blks.N. int. of business rt. and regular US 59	10" of caliche base Subgrade soil
73	" "		
74	No. 4, St. 202	3 Mi.E. of Beeville	7" of caliche base Subgrade soil
75	" "		

Texas Highway Department
Form 473A 7-58-5m

84005-758-5m

SOILS AND BASE MATERIALS TEST REPORT

Laboratory No. _____
Date Rec'd _____ Reported _____
Engineer _____
Address _____
Contractor _____
Sampler H. S. Gillette & C. McDowell
Sampler's Title _____
Sampled From Roads
Producer _____
Quantity Represented by Sample _____
Has been Used on _____

Control Number | Section Number | Job Number
County | Federal Project No. | Highway No.
District No. | L.P.E. No. | Req. No. | Date Sampled
Specification Item No. _____
Material from Property of _____
Proposed for Use as _____

Lab. No.	LL	PI	FME	CME	SL	LS	SR	Class	Soil Binder	% Moist.	Str. Class
50-82-E	34	20	22	-	16	9.3	1.91	A-6-2	27	-	3.1
83	24	8	18	-	17	4.0	1.79	A-2	96	8.2	3.1
78	31	14	22	-	18	6.8	1.82	A-2	36	-	2.8
84	82	56	49	-	12	25.8	2.05	A-7	86	23.6	5.2
80	29	15	21	-	16	6.8	1.87	A-2	31	--	2.7
81	66	41	39	-	15	20.2	1.90	A-7	96	24.6	4.7
49-40-R	24	7	20	-	17	4.0	1.78	A-2	32	--	3.0
39-11-MR	74	45	45	-	12	24.0	2.00	A-7	97	31.	4.9

PERCENT RETAINED ON

Lab. No.	Round Opening Screens: Opening in Inches 2½	2	1½	1	¾	½	¼	Square Mesh Screens: Sieve Numbers 10	20	40	60	100	200	Grain Diam. in Millimeters .05	.005	.001	Specific Gravity
50-82-R			3	7	19	27	40	56	67	72	73	76	79	83	85	92	97	2.70
83							1	2	3	4	7	21	63	69	83	92		2.60
78				8	19	28	37	51	59	62	64	66	67	69	71	90	98	2.65
84						0	5	8	12	13	14	14	16	22	27	61	83	2.66
80				3	14	22	34	47	59	64	69	75	80	84	86	98	99	2.75
81									1	3	4	5	7	16	20	63	83	2.71
49-40-R				8	17	21	28	38	51	62	68	73	82	90				2.60
39-11-MR											3	3	4	5	12	54	77	2.71

SAMPLE IDENTIFICATION

Lab. No.	Identification Marks	Location—Properties—Station Numbers	Type of Materials
50-82-R	No. 5 St. 346	Bexar Co. Sta. 888,12 Mi.S.Int.Loop 13 and 346	13" Gravel Base Subgrade Soil
83	" "		
50-78-R	No. 6 Loop 13	2000' N.Main gate,Lakcland Air Base	13" Gravel Base Subgrade Soil
84	" "		
50-80-R	No. 7, Loop 13	2000' E. of Kelly Field Overpass	12" Gravel Base Subgrade Soil
81	" "		
49-40-R	US 290 Travis Co. East of Manor		10" Gravel Base Subgrade Soil.
39-11-MR	" "		

Texas Highway Department
Form 473A 7-58-5m

84005-758-5m

SOILS AND BASE MATERIALS TEST REPORT

Laboratory No. _____
Date Rec'd _____ Reported _____
Engineer _____
Address _____
Contractor _____
Sampler H. S. Gillette & C. McDowell
Sampler's Title _____
Sampled From _____
Producer _____
Quantity Represented by Sample _____
Has been Used on _____

Control Number | Section Number | Job Number
County | Federal Project No. | Highway No.
District No. | L.P.E. No. | Req. No. | Date Sampled
Specification Item No. _____
Material from Property of _____
Proposed for Use as _____

Lab. No.	LL	PI	FME	CME	SL	LS	SR	Class	Soil Binder	% Moist.	Str. Class
50-90-R	35	18	23	-	18	8.5	1.76	-	48	-	3.1
91	34	19	22	-	16	8.7	1.86	A-7-4	100	9.2	4.6
92	35	17	26	-	20	7.6	1.75		59	-	3.4
93	36	20	25	-	15	10.5	1.87		100	10.6	4.4
49-14-R	22	6	19	-	15	4.1	1.89		26	-	1
47-137-E	30	7	23	-	25	2.7	1.59	A-2	98	-	3.4

PERCENT RETAINED ON

Lab. No.	Round Opening Screens: Opening in Inches 2½	2	1½	1	¾	½	¼	Square Mesh Screens: Sieve Numbers 10	20	40	60	100	200	Grain Diam. in Millimeters .05	.005	.001	Specific Gravity
50-90-R				0	7	19	24	30	37	44	49	52	54	63	77	80	90	98	2.64
91												3	10	22	39	78	92		2.68
92					2	5	8	13	24	33	38	41	43	50	63	70	88	97	2.65
93													2	11	27	48	78	92	2.65
49-14-R					13	25	31	39	48	56	65	72							
47-136-E											0	2	5	53	74	81	84	87	2.63

SAMPLE IDENTIFICATION

Lab. No.	Identification Marks	Location—Properties—Station Numbers	Type of Materials
50-90-R	No. 14 US 70	Floyd Co., 4 Mi.E. of Hale Co.Line Sta. 330+15	9" Caliche Base Subgrade Soil
91	" "		
50-92-R	No. 15, US 62	Crosby Co. 1 Mi. S. Floyd-Crosby Co. Line	7" Caliche Base Subgrade Soil
93	" "		
49-14-R	US 290	8" Crushed Stone Base	Flexible Base Subgrade Soil and Select Material
47-136-E	"	Sampled from R.O.W. Cut	

for design thicknesses. Figure 11 indicates that good long-life pavements may be approximately one and one-half inches thinner than is required by our design procedure; therefore, in construction of new projects we often reserve an inch or two of surfacing for future application. It may also be noted that plotting points Number 2 and 4 are very close together on this chart although one is from a fairly good road and the other from a poor road, the difference being that the poor road is twenty years old and the fairly good road is only three years old. Figure 12 shows the relation of road age to "per cent design" expressed as 100 (ratio of existing depth to depth required by triaxial tests minus one and one-half inches). This chart separates plotting points 2 and 4 more nearly as should be. In fact, all points tend to be arranged so that the line represented by the expression, Number years life = $0.96\ (10)^{.01465\ (\%\ design)}$, divides points for road failures on the left from points for good and excellent roads on the right of the line. The results shown are limited; however, for the time being they will help explain why some underdesigned pavements are not failing rapidly. It is also believed that these relations will help designers to select design thicknesses which are in keeping with engineering and economical needs whether for a short-life road requiring relocation soon, for stage construction, or for very long-life urban sections. It is doubtful that the term "per cent design" is of much value in estimating the life of bituminous surfacings (existing on long-life bases) which are less than one and one-half inches thick because the triaxial method does not measure the properties of asphalts. Since the amount of maintenance required for roads is somewhat dependent upon skill and timing of operations, it is by no means certain that projects having similar percentages of design will require the same amounts of maintenance. General observation of some of these projects throughout their life bears out this statement.

From the foregoing it is concluded that:

1. There is a good correlation between the Texas triaxial method of pavement thickness design, stress calculations (using the theory of elasticity), and actual road performance.

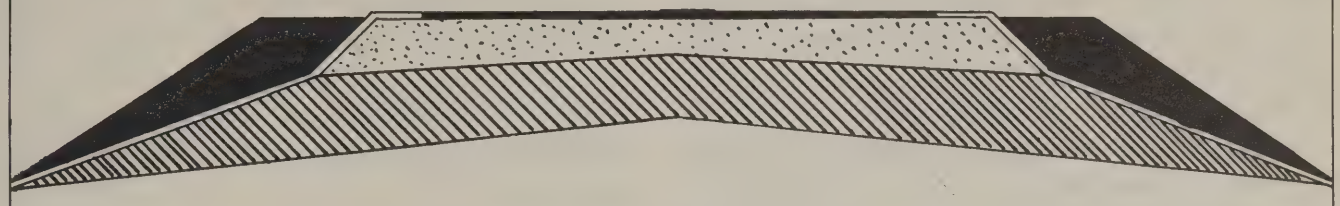
2. Since the data obtained from triaxial tests made it possible to correlate "per cent design" with life of pavements, it appears that triaxial methods can be used successfully to analyze most highway flexible pavement problems. In addition to estimating over-all thicknesses on subgrades, the method also determines the quality and density requirements of subbase and flexible base materials, the latter being equally, if not more, important than over-all thickness because good pavements are a prerequisite to proper thickness evaluations. The term "per cent design" appears to be a basic expression and is necessary in order to have a good understanding of flexible pavement design.

2. The simplicity and workability of the method are attested to by the fact that over one-half of our district offices either have been or are being equipped for triaxial testing. This program has been carried out voluntarily by the districts, and is not mandatory.

3. We have progressed a lot in the analysis of this problem in the last several years, but much still remains to be done.

Acknowledgements: The writer is indebted to many who have contributed, encouraged, or assisted in the development of triaxial testing for use in the design of pavements. A few of these people or organizations have been mentioned in the text; however, it should be mentioned that the work of the members of the Soils Section and other members of the Materials and Tests Division of the Texas Highway Department, under the guidance of Mr. A. W. Eatman, has been a major factor in making this report possible. The author is also indebted for assistance received from personnel of the Fort Worth office of the Bureau of Public Roads, and the Road Design and Planning Survey Division of the Texas Highway Department.

DESIGN OF CROSS SECTION AS



INFLUENCED BY EDGE FAILURE PROBLEMS

A.C. Kyser Jr., Assistant District Engineer
Texas Highway Department
Houston, Texas

EDGE FAILURE PROBLEM: The simplest solution to this problem would be to keep the wheel away from the edge, if such a solution were feasible. The large number of edge failures in the flexible base sections constructed in the Gulf Coast area on clay-type subgrades warrants some very serious study of this problem. There are miles of farm-to-market roads constructed since the war on which the center sections appear to be in good condition, but which show numerous failures and patches along the edge. One particular road looks as though it was originally constructed about 9 feet wide and widened on both sides to an approximate width of 18 feet. This road has been in service about two years. It is obvious that it would have been more economical to have designed the cross section originally to prevent this expensive maintenance.

For this paper, the discussion refers to flexible base construction on a highly plastic clay subgrade in the Gulf Coast area.

BRIEF REVIEW OF DESIGN PROCEDURE: The recommended design procedure involves the testing of disturbed samples prepared in the Laboratory and subjected to wetting and drying cycles in a prescribed manner, and then testing in the triaxial cell to determine soil shearing strength. From the results of the triaxial test, the soil is classified. The class of soil and wheel load determines the

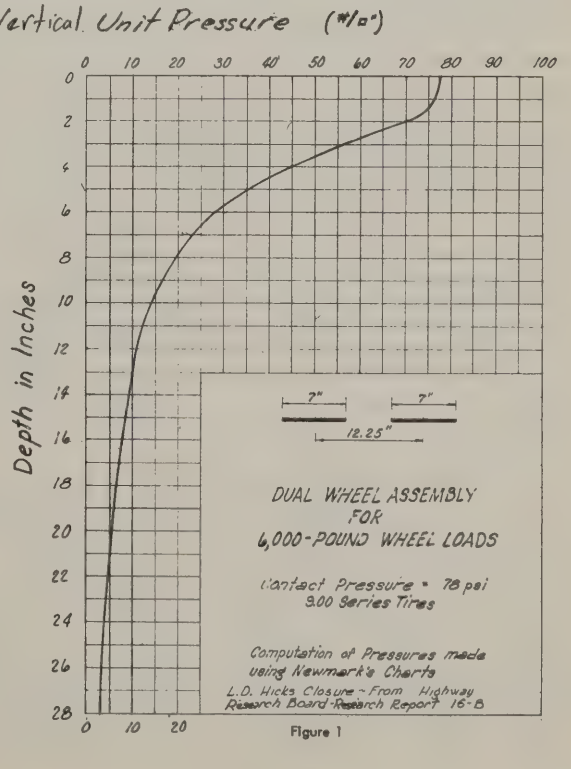

Vertical Unit Pressure (#/□")
0 10 20 30 40 50 60 70 80 90 100
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28
Depth in Inches
0 10 20
7"
7"
12.25"
DUAL WHEEL ASSEMBLY
FOR
6,000-POUND WHEEL LOADS
Contact Pressure = 78 psi
9.00 Series Tires
Computation of Pressures made using Newmark's Charts
L.D. Hicks Closure - From Highway Research Board Research Report 16-B

Figure 1

depth beneath the surface that such a soil can be used.

You will see in Figure 1 a curve showing the vertical stress in the flexible base of subgrade material as plotted against depth below the surface. This particular curve was prepared for dual 6,000-pound wheel loads, or a total axle load of 24,000 pounds. You will note how the vertical stress decreases with the depth. Shearing stresses decrease in like manner. After the strength of the material, either flexible base, selected roadbed treatment, or subgrade is determined, curves of this type, or equivalent curves plotted on a Mohr's diagram, will show the minimum depth beneath the surface that each material can be used. Similar curves have been prepared for other wheel loads.

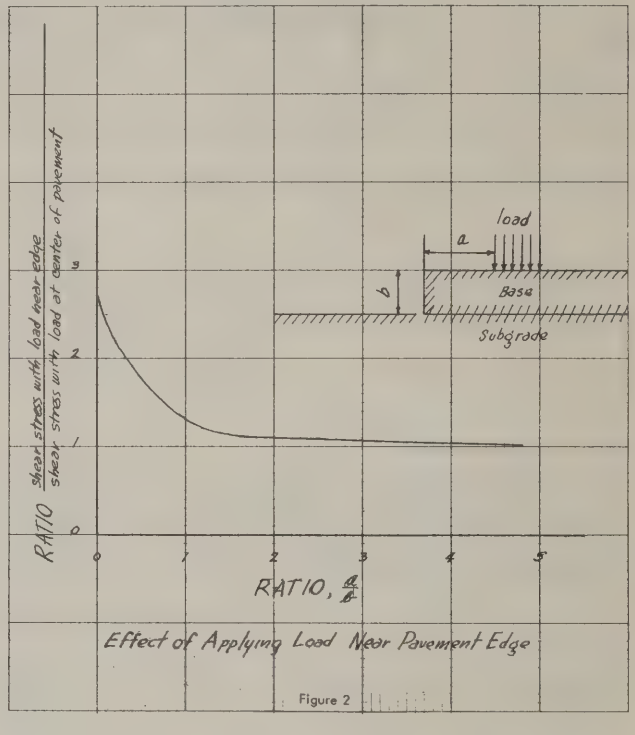
This rational method of design is considered the best that is now available, except for the fact that this procedure does not automatically take into account the cross section of the pavement. Depth alone is not the sole factor. I quote Mr. W. S. Housel, Professor of Civil Engineering, University of Michigan, in General Discussion of Flexible Base Pavements:

"While the attempt to establish a numerical rating for a project which includes a systematic evaluation of each and every factor which affects pavement performance is to be commended, the result obtained raises a number of questions. As far as the writer is concerned, the mechanics of the pavement structure appear to have been so submerged by numerical ratings and coefficients that the result is no longer definitive of structural behavior. Insofar as pavement performance is concerned, the complex numbers convey no mental picture of the actual condition of the pavement. In other words, the perspective of the entire problem and the relation between its component parts seems to have been lost."

Mr. Housel did not make particular reference to stresses occurring at the edge, but his comment seems to be applicable.

REASONS FOR EDGE FAILURES: The principal reasons for edge failures are the increased stresses occurring at the edge of the pavement, the lack of lateral support, and the decrease in strength of the subgrade caused by infiltration of water.

At my request, Mr. Frank Scrivner of the Road Design Division made an approximate computation to show the stresses in the subgrade caused by changing the wheel location with reference to edge of the base. (See Figure 2.) Although for purpose of computation it was necessary to make certain simplifying assumptions, the relationship of the stresses is considered to be approximately correct.



Effect of Applying Load Near Pavement Edge

Figure 2

According to this graph, you will see that the stress in the subgrade when the wheel is at the pavement edge is approximately two and one-half times the stress when the wheel is at a distance two or three times the depth of the base from the edge.

The other important factor concerns the drying out of the shoulder and subgrade material in dry seasons and the subsequent wetting of the pavement subgrade after heavy rains. I am sure all of you have seen wide cracks open at the pavement edge during the summer. These cracks form a channel for feeding the water into the subgrade. A clay soil in a dry condition will absorb a sufficient amount of water to lower the structural strength considerably below that of the same material constructed to Proctor Density at

Optimum Moisture. Heavy loads at the pavement edge cause subgrade displacement and resulting depression. This depression forms a reservoir for collecting rain water, thereby feeding this water into the subgrade through cracks that occur in the surface and base because of the failure. Consequently, progressive failure is accelerated, as depicted in this figure.

The Western Association of State Highway Officials' road test in Idaho has indicated the need of shoulder protection to prevent the type of edge failure illustrated in Figure 3.

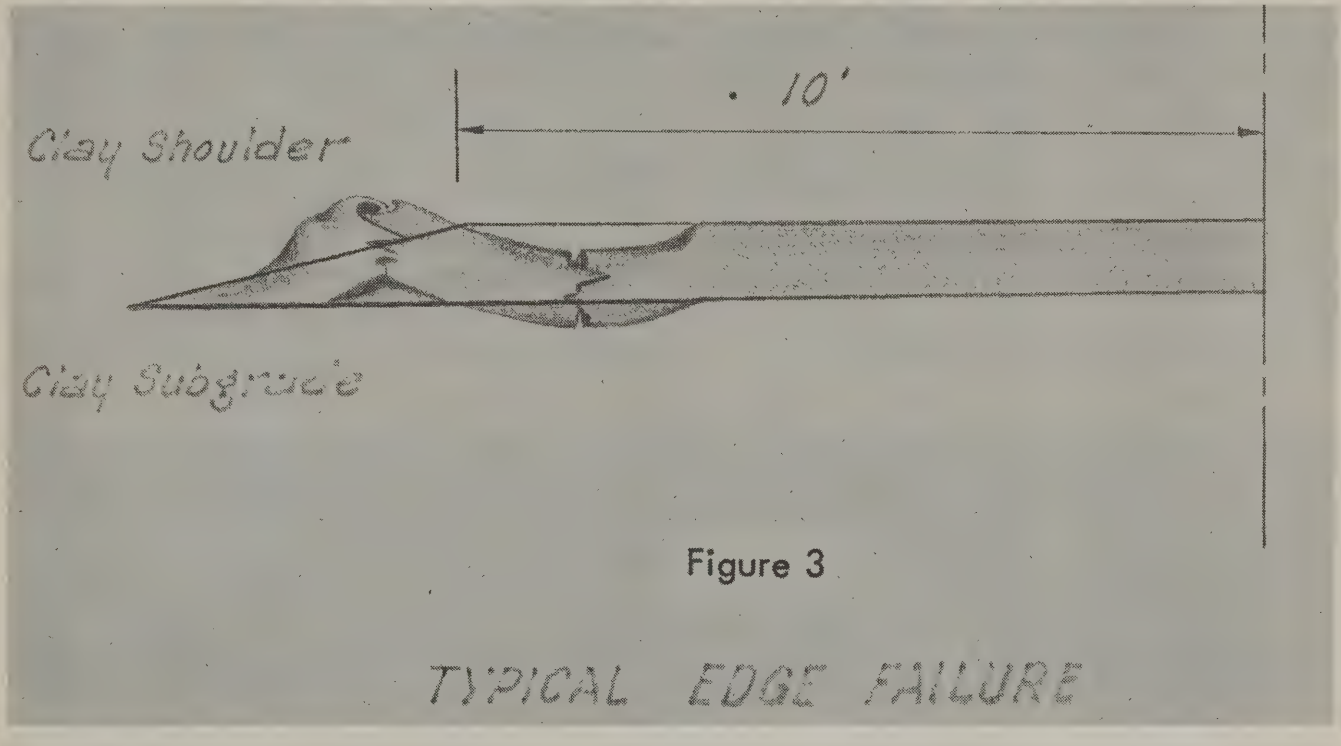


Figure 3

TYPICAL EDGE FAILURE

Quoting from the report presented to the 33rd Annual Meeting of the Highway Research Board, January 12, 1954:

"There was a striking difference in the behavior of that part of the pavement near the edge (outer wheel path) and that near the centerline (inner wheel path). In all cases of failure, the distress was first noticed in the outer wheel path and in most cases no failure whatever was noted in the inner wheel path even months after the outer wheel path had failed completely. Subgrade moisture content and density tests do not supply an adequate explanation for the radical difference in behavior of the two wheel paths. Devices installed along the pavement edge showed that under traffic there was appreciable shifting of the granular base material outward toward the shoulder during the period when failures were occurring. This lateral movement of base material stopped at the same time that pavement failure stopped progressing. The excellent behavior of the pavement in the inner wheel path led the Committee to add paved shoulders to one test section in July 1953. It is hoped to determine whether or not this treatment is beneficial during the critical period next spring."

Later reports from the WASHO test indicated that the paved shoulders were very effective in preventing edge failures.

SUGGESTED REMEDIES: I will enumerate briefly four possible remedies to prevent edge failures:

1. Application of an asphalt membrane and stabilized shoulders to prevent excessive moisture changes in the subgrade, and add lateral support.
2. Stabilizing the subgrade with lime.
3. Adding a blanket of selected local materials as roadbed treatment.
4. Increasing the width and depth of the base course.

USUAL LOW TYPE FLEXIBLE BASE SECTION: Figure 4 shows a section used on numerous projects in District 12 immediately after the war. We were given a total allot-

ment of funds and the number of miles of road was more or less specified. This left insufficient funds to construct a section of adequate width and depth to accommodate the anticipated traffic, especially in areas where the subgrade was predominantly clay.

For purposes of comparing the costs of the various remedies suggested below, these estimates are based on the expenditure required to construct the section over and above the section shown here as the minimum. In other words, for sections showing increased depth and width, only material in the widened or deepened portions is included. None of these suggested remedies is considered to be the cure-all in itself, but perhaps a combination of these should be used depending on the availability of local materials.

SHOULDER TREATMENT: The section of Figure 5 is based on an asphalt membrane of one-half gallon per square yard with sand or select topsoil shoulders. The purpose of this treatment is to prevent drying out of the subgrade and to add lateral support.

in 1940 on U.S. 90 between Sealy and Brookshire, an asphalt membrane consisting of one gallon per square yard of OA-90 was used on selected sections. The membrane extended for the full width of the subgrade crown, and shoulder material of bank-run river gravel was placed on top of the membrane. It is obvious that the membrane was effective in controlling the moisture because there was no sod growing on the sections that were undersealed with the membrane, whereas on the adjoining sections there was considerable vegetation. This section was cored recently to determine whether the asphalt membrane was still effective in controlling moisture after fourteen years. This was in a dry season and, unfortunately, we have not had an opportunity to compare the results with the moisture content in a wet season for this section. The moisture content immediately below the membrane was 22 per cent, whereas in the adjacent section that did not have the membrane, the moisture content was 10 per cent or less for a depth of 3.5 feet.

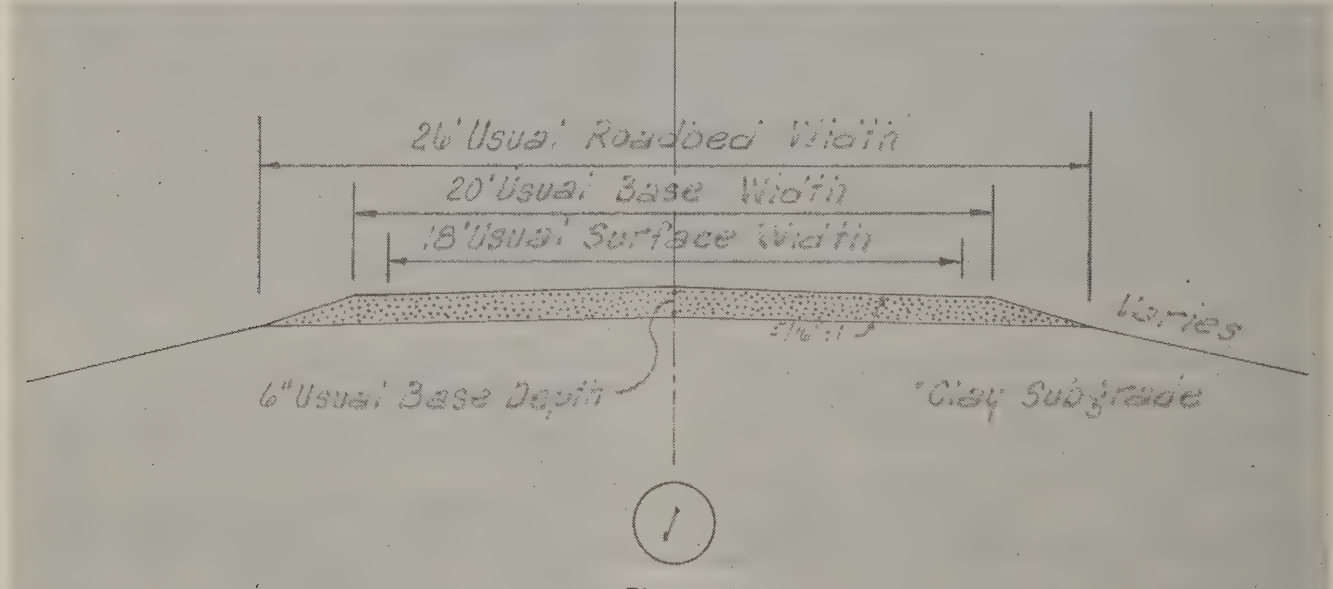


Figure 4

TYPICAL LOW TYPE FARM TO MARKET ROAD

There has been considerable discussion as to the durability of an asphalt membrane. In fact, there are some that have insisted that an asphalt membrane used as waterproofing would become fertilizer within a few years. In the construction of a concrete pavement

It is reasonable to expect that after a period of heavy rains the moisture content in the sections that are now dry will rise considerably higher than that in the sealed sections and the subgrade strength will decrease correspondingly. A core showing this asphalt

seal is on hand for visual inspection of those interested.

The Houston Urban Expressway organization, under the direction of Mr. Van London, has followed a practice of wrapping the fill sections in an asphaltic membrane on the top, bottom, and sides. The membranes on the sides are placed on a one and one-half to one slope. The membrane has been effective in controlling moisture in the fills as proven by numerous tests taken since the installation of the first section about seven years ago. It was noted that two years ago, during an exceptionally dry season, there were cracks on the slopes about two inches wide. These cracks did not extend into or damage the membrane, and the original moisture was retained in the fill. For further reference in regard to the use of asphalt membranes on the Gulf Freeway in Houston, you are referred to Roads and Streets magazine, April, 1953, issue. Asphalt membranes on shoulder slopes are currently being used on several rehabilitation projects in the Houston District.

LIME STABILIZATION: Laboratory tests indicate that when a clay is mixed with lime (see Figure 6), its behavior and the Plasticity Index resemble a granular material. In addition, there seems to be considerable cementing action which gives a strength even greater than the strength of most roadbed treatment materials. For a recent project using lime stabilization, the PI was reduced from 40 to 8 in the Laboratory and in construction, a maximum PI of 12 was obtained.

We were recently given approximately 35,000 cubic yards of waste lime by Wyatt Metal and Boiler Works in Houston. This lime came from land previously owned by a chemical plant and was furnished to the State without cost on condition that it be removed within two years. This material is now being used successfully on a farm-to-market road southwest of Alvin. Based on experience to date on this project, which involves a maximum haul of 45 miles, it is estimated that a ten-inch blanket of lime stabilized subgrade to accommodate a 24-foot base width costs approximately $11,000 per mile which is equivalent to $2 per cubic yard of compacted material.

The section shown in Figure 7 is similar to that previously shown. Roadbed treatment or special treatment of the clay-type subgrade should be used if it proves to be cheaper than flexible base, which is usually the case. In the event that the funds allotted a project are insufficient to construct both flexible base and roadbed treatment to depths indi-

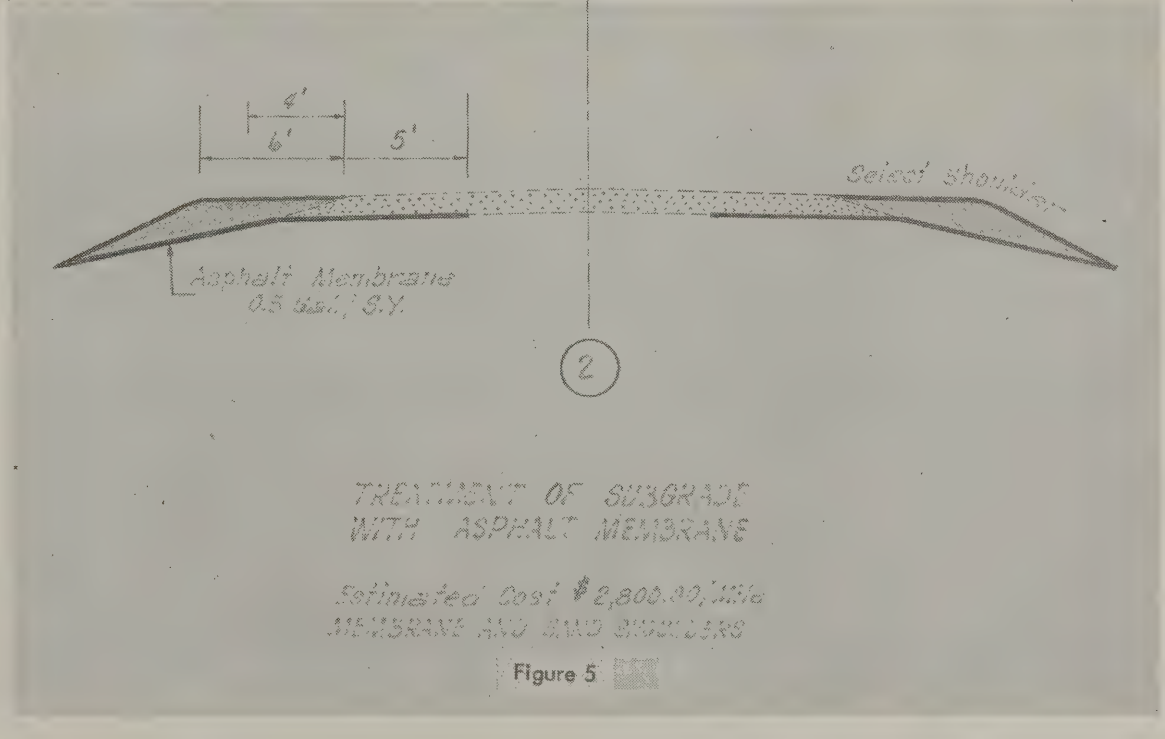
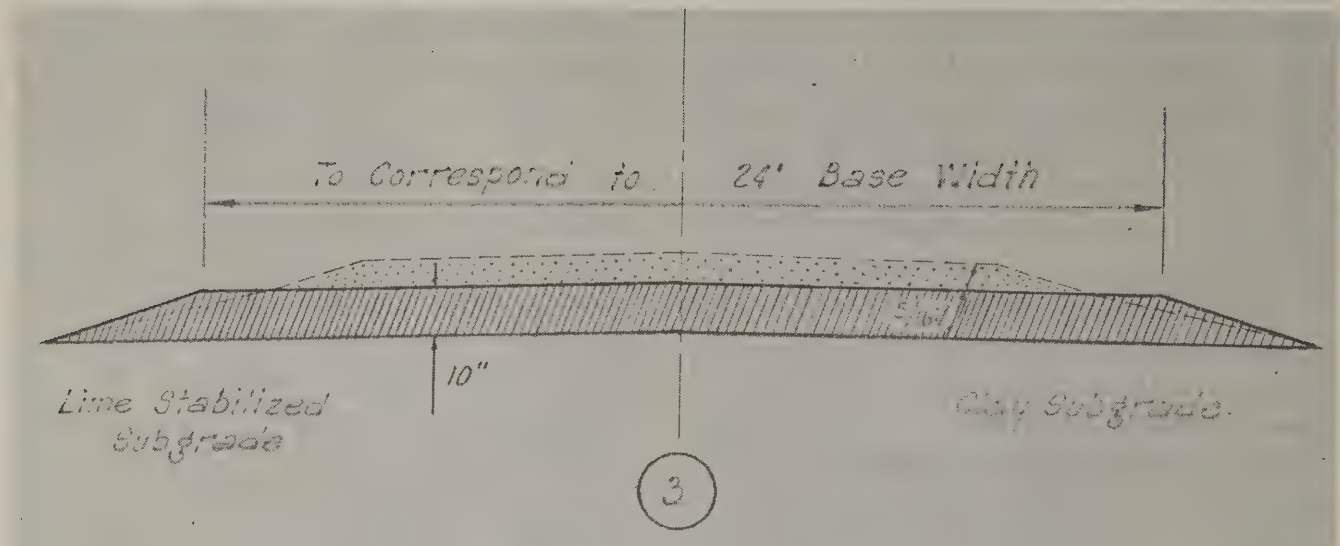


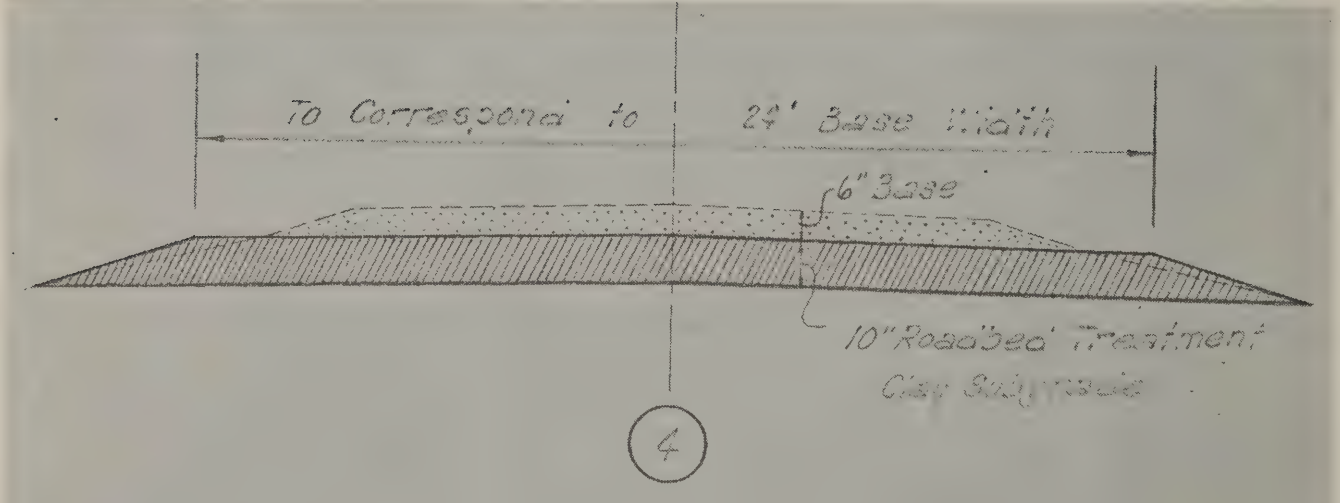
Figure 5



LIME STABILIZATION

Estimated Cost $2.00/S.Y. Treated Subgrade
Estimated Cost $11,000/Mile Treated Subgrade

Figure 6



ADDITION OF ROADBED TREATMENT

Estimated Cost $10,000/Mile

Figure 7

cated by design tests, the more expensive flexible base should be reduced in depth because the roadbed treatment is a better buy in that you get more depth for the money.

In Figure 8, the section shows the treatment of the problem by increasing the depth and width of the base. The base is shown as uniformly ten inches thick. The extra cost of this flexible base material computed on average prices in our area is $12,000 per mile.

base section including the sealing and stabilization of the shoulders. This section is designed to accommodate dual wheel assemblies for a 6,000-pound wheel load, 24,000-pound axle load, where the shearing strength of the subgrade is approximately Class 5.7 and the shearing strength of the select material or roadbed treatment is Class 3.7. (See design chart, THD - 80, 10 year life.)

The base width corresponds to a 20-foot

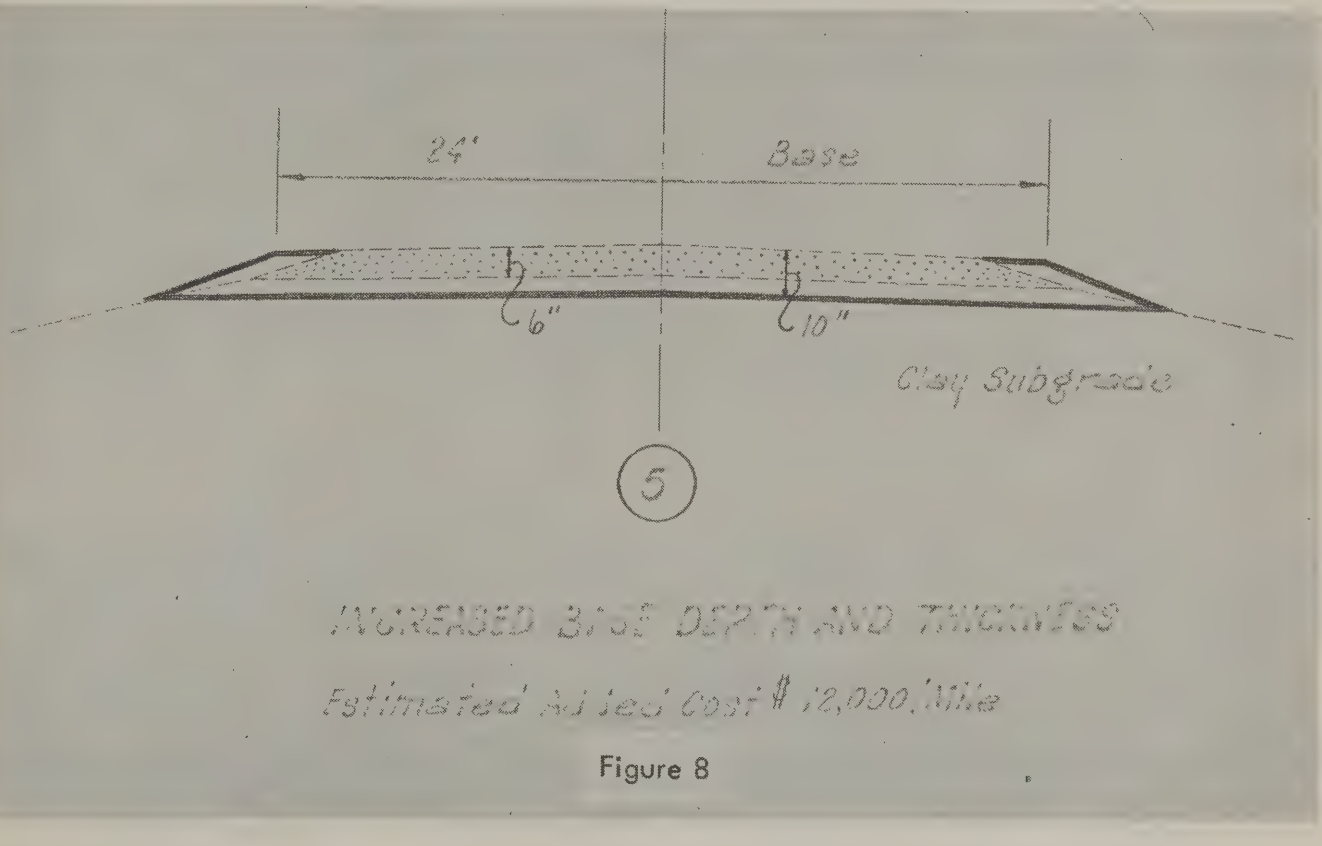


Figure 8

Figure 9 is a modification of the previous section to show the increased depth on the edge without appreciably increasing the depth at the center. The extra cost of material shown on this section is approximately $9,000 per mile, which is 75 per cent of the cost of section previously shown. Inasmuch as the failure usually occurs on the edge, it is quite possible that pavement constructed as shown on this section would give equal service to the one shown previously, resulting in a saving of $3,000 per mile.

Figure 10 shows the desirable section and is a composite of the roadbed treatment section previously shown, the 10-7-10 flexible

surface width. For 24-foot surfacing a 28-foot base width is considered the minimum and full crown width of base is desirable. The depths referred to in this paper are not to be taken as recommended depths. As pointed out previously, the depth of flexible base should be determined by testing the quality of the roadbed treatment to find how near the surface this material can be used satisfactorily, and the depth of roadbed treatment should be determined by testing the subgrade to ascertain the over-all thickness of flexible base and roadbed treatment. In many cases, funds will not be sufficient to construct the recommended sections based on the designs as shown, and in such cases it

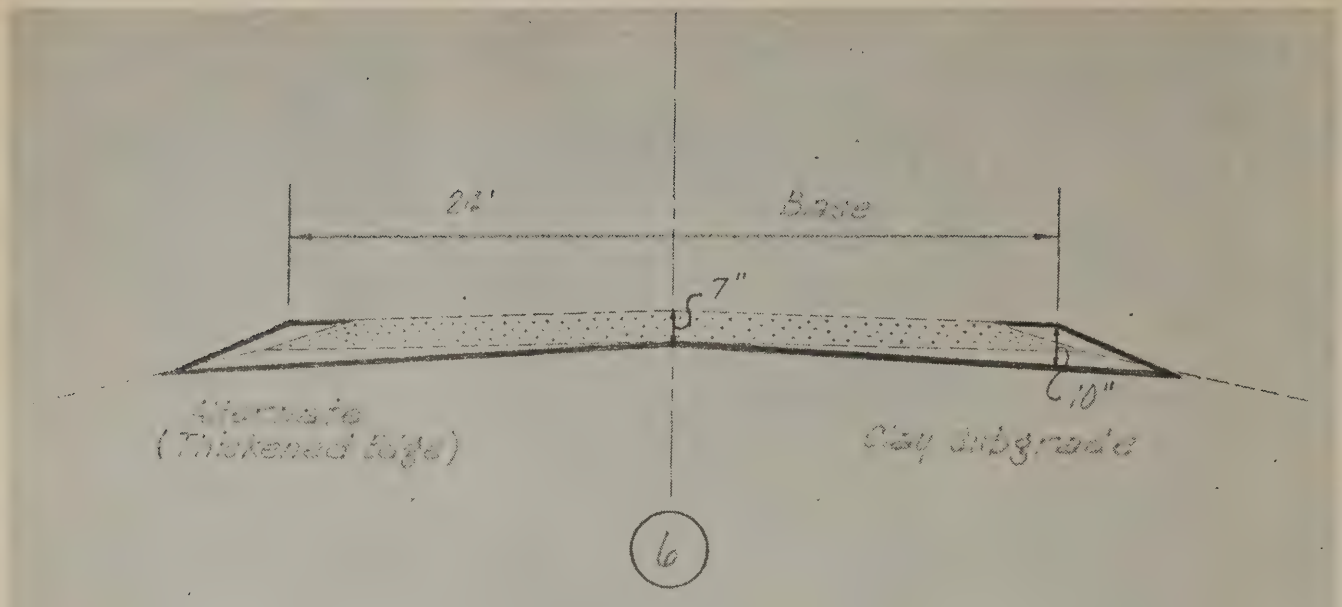


Figure 9

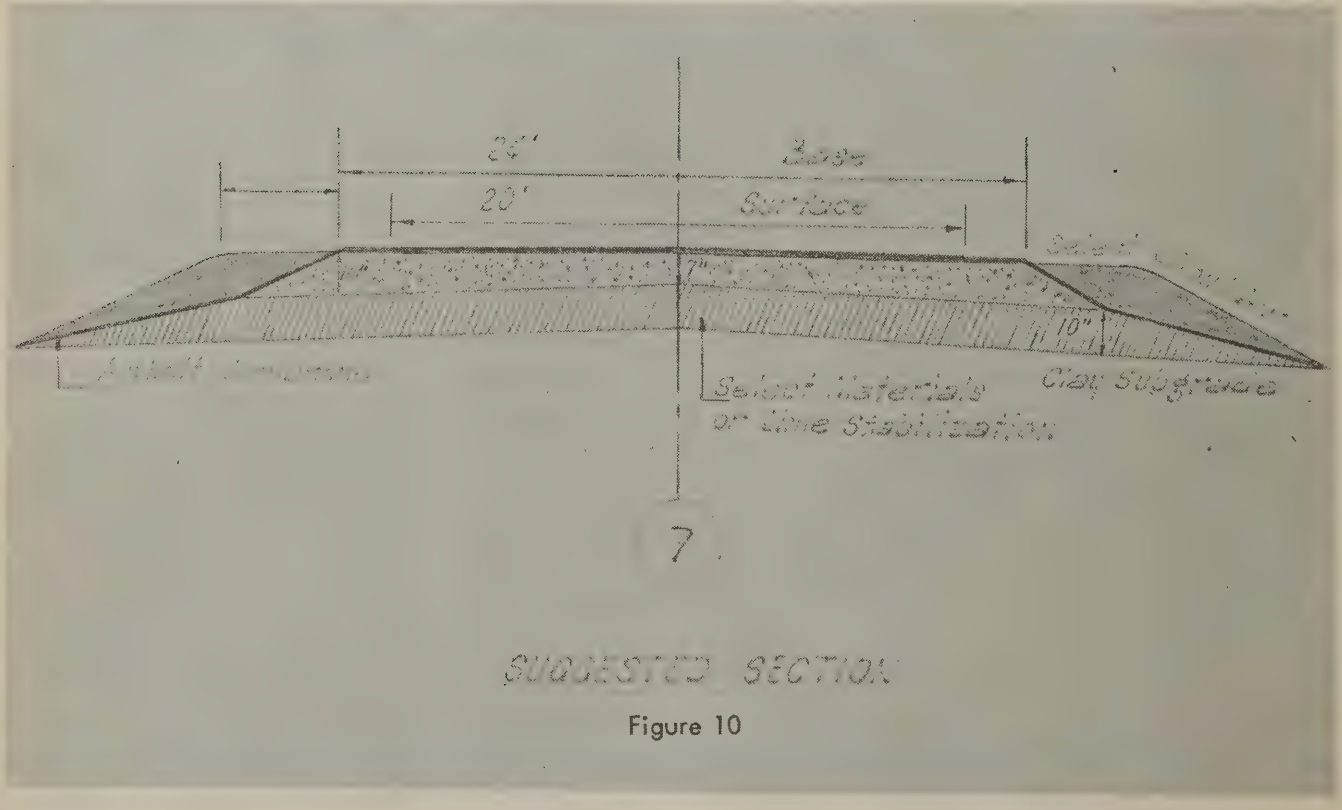


Figure 10

is recommended the sections be built by stage construction, and the depth of flexible base be reduced to the minimum amount necessary without reduction in either the depth of roadbed treatment required or elimination of the asphalt membrane on the shoulders.

STRUCTURES AND HYDRAULIC SECTION

Morning: Room 2-B

Chairman: H. K. Stephenson, Research Engineer
A & M College of Texas

9:00 Use of Lightweight Aggregates in Structural Concrete
William Shafer, Senior Concrete Engineer
Texas Highway Department
Austin, Texas

Cedric Willson, Vice-president
Texas Industries, Inc.
Dallas, Texas

10:30 Possibilities of Precast Concrete in Highway Structures
Truman R. Jones Jr., Assistant Research Engineer
A & M College of Texas

Afternoon: Room 2-B

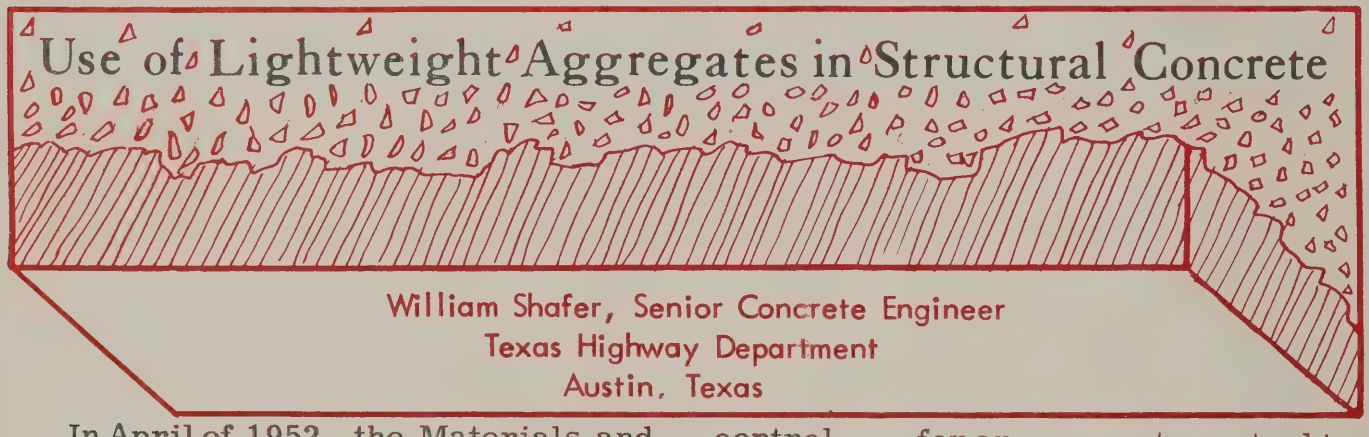
Chairman: M. D. Shelby, Senior Soils Engineer
Texas Highway Department
Austin, Texas

1:30 Current Design Methods for Drilled Foundations
Frank S. Mitchell Jr., Laboratory Engineer
Texas Highway Department
Houston, Texas

3:00 Results of Load Studies on Drilled Shafts
L. A. DuBose, Assistant Professor
A & M College of Texas

The use of lightweight aggregates in structural concrete is not a recent or unknown procedure. A concrete ship, the Selma, was constructed using reinforced lightweight aggregate concrete, and operated during World War I. However, perhaps due to lack of promotion supported by adequate research, the use of lightweight aggregate concrete has not come into its own. Recently, there has been an increased interest evidenced by both State and Federal agencies and also by private construction in the use of lightweight aggregate concrete for structural use.

The author has investigated three different sources of lightweight aggregate for use in structural concrete. Each one of these investigations was originated by different districts within the Highway Department with the intention of the possible solution of various problems.



Use of Lightweight Aggregates in Structural Concrete

William Shafer, Senior Concrete Engineer
Texas Highway Department
Austin, Texas

In April of 1952, the Materials and Tests Laboratory of the Highway Department received a request from District 12 at Houston relative to design of Class "A" concrete in which expanded clay from the Herculite Corporation of Houston was to be used as both fine and coarse aggregates. The problem confronting District 12 was one primarily of design of lightweight aggregate concrete. As a means of an introduction to this problem of design a quotation is taken from "Field Practice in Lightweight Concrete," by John A. Murlin and Cedric Willson:

"The absorption of clay and shale aggregates is an extremely elusive and variable property. As a matter of fact, after working on absorption for three years, the only thing of which we are certain is that this property, a very minor factor with natural heavy aggregates, is extremely important in dealing with expanded clay and shale aggregates. After making hundreds of tests, we believe the true absorption of expanded shales to be from 7 per cent for coarse aggregates to 20 per cent for the 'fines' or sand grade, and expanded clay aggregates to be from 16 per cent to 26 per cent for the coarse and fines. However, we are far from certain as to the accuracy of these determinations.

"We also found the specific gravity of these materials to be practically indeterminate due, of course, to the variable absorption. The specific gravity apparently varies with the particle size. Attempts were made to determine the specific gravity in many fluids other than water but results were not satisfactory.

"It is now necessary to develop a simple workable method of design and control --- for any concrete control to be practical the operations at the mixer or plant must be the same for each batch. In order to produce uniform concrete, it was necessary to base the design and control on aggregate properties which were constant and stable. It has been found that the only condition in which expanded clays and shales were constant and unchanging was when they were moisture free. Mixing water was added by trial until the desired slump was obtained. It was found this same amount of water would continue to produce the same slump on that day from the particular stockpile of aggregate. After a little experience the control technicians would invariably get the correct amount to add by the second or third batch and it is the only simple method we were able to devise by which the water-cement ratio was uniformly

maintained. To sum up this method of design and control it was found that the dry material was constant only when moisture free so that condition was used for the basic weight or volume of aggregates. It was also found that, of the total amount of water required, the only constant fraction was that part which was not absorbed by the aggregate. This made it necessary to measure that fraction by maintaining a uniform slump. The slump cone test for consistency was found to be much more sensitive for expanded clay and shale than for heavy concrete."

The above quotation was found to be true in every respect. An attempt was made to determine the true specific gravities on Haydite lightweight aggregate through the use of varying periods of immersion. As these immersion periods were in progress, periodic checks were made on the weights of the pycnometers containing the specific gravity samples. When these weights became constant the gravities were then calculated. Another sample was immersed in water for 24 hours and then put into a pressure pycnometer operating at 1,200 p.s.i. These specific gravity determinations would check one with another, but when they were used to design a batch of lightweight aggregate concrete, the concrete yield was in error as in previous trials. Due to this difficulty experienced here in the selection of an acceptable concrete design, the following procedure is recommended.

The accepted method of concrete design outlined in Construction Bulletin C-11 is based on absolute volumes; quantities are measured by weight, making use of the fixed relation between weight and volume which we call saturated-surface-dry specific gravity. In lightweight aggregates the variable absorption makes this SSD gravity an indeterminate quantity. Therefore, the customary design procedure simply is not compatible with the characteristics of the aggregate and some other procedure must be devised.

The method that is recommended here involves pilot tests, employing a full-sized miner and full-sized trial batches on the job. The method of design is outlined below:

Determine the dry unit weights per cubic foot of both the fine and coarse aggregates. This is necessary, not as a step in the design, but to conform to A. S. T. M. Designation C 130-42 which sets a limit on the weight per cubic foot of both the fine and coarse aggregates. The weight per cubic foot of the aggregates may also be used, after the design is determined, to relate the mix to a volume basis.

The approximate absorption and/or free moisture of the materials may be determined so as to reduce the assumption of total water necessary to produce the required workability. If the absorption is to be determined the material should stay submerged in water at least 72 hours prior to the determination.

In determining the batch which will provide a suitable yield and workability, start with a combined aggregate weight which will produce a lower yield than that which is intended. In convenient increments, increase this total aggregate weight until a higher yield is produced than intended. The amount of water on each batch should be controlled by slump tests so as to produce the required workability. Test specimens may be taken from each batch run; curves may be constructed reflecting strengths vs. water-cement ratio and yield vs. total pounds of aggregate used. From these curves drawn on the pilot batches a suitable design may be obtained for use on the job.

The weight of lightweight aggregates, tabulated in Table 1, found necessary to produce a suitable concrete batch yielding 27 cubic feet of concrete per cubic yard, are for all three sources of lightweight aggregates which were investigated, namely, Haydite expanded clay of Houston, Texas; expanded shale from San Antonio, Texas, Converse Pit; and expanded shale from Strawn, Texas.

The flexural strength at seven days and the compressive strength at 28 days that could be anticipated when using lightweight aggregate for

structural use were also under investigation. It was found when a lightweight aggregate concrete batch is properly proportioned producing a workable plastic mixture containing 27 cubic feet and 6 sacks of cement per cubic yard of concrete, that the flexural and compressive strengths would compare favorably with the conventional Class "A" concrete now in use by the Highway Department. There is one point that should be made at this time. Concrete produced using expanded shales and expanded clays as their aggregates will attain approximate maximum flexural strengths when tested at the end of a 40-day curing period. The approximate maximum compressive strengths are obtained when tested at the end of a 60-day curing period. Flexural strengths, when tested at seven days, are approximately 77 per cent of 40-day flexural strength. Compressive strengths, when tested at 28 days, are approximately 78 per cent of 60-day tests. These comparisons, when taken on a percentage basis, agree very closely and are significant. In Bar Diagram No. 1 are listed the flexural and compressive strengths which are typical of concrete made from the lightweight aggregate sources investigated in the Materials and Tests Laboratory.

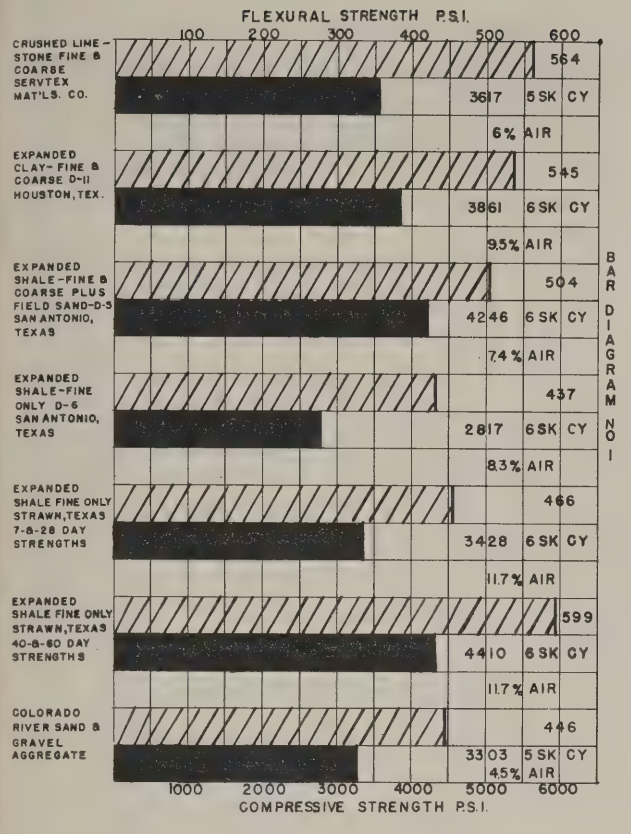
RETROGRESSION OF FLEXURAL STRENGTH: On a construction project a test beam inadvertently was permitted to dry out for about a week following the moist curing period and was tested in the dry condition. Its flexural strength was approximately half that of the moist-cured beams previously tested. The question arose as to whether or not this was a characteristic of lightweight aggregate concrete. The method of investigation consisted of preparing a number of beam specimens for flexural tests using lightweight and conventional aggregates and employing various methods of curing. There were in general, two groups of beams. Specimens from one group were continuously moist cured for various periods and tested while moist; specimens from the second group were tested in a dry condition following seven days of air drying, with or without preliminary periods of moist curing. Beams that were tested immediately following seven days of air drying consistently had much lower flexural strengths than the beams that were tested immediately following periods of moist curing regardless of the type of aggregates. In order for test results to be significant, it is necessary that the method described in A. S. T. M. C 42 be adhered to.

It is postulated by the writer that when a concrete specimen dries, tensile stresses are set up in its outer portions. When a flexural load is applied to such a specimen, the stresses already present are in the same direction as the applied stresses and tend to decrease the applied load necessary to rupture the specimen.

MODULUS OF ELASTICITY: In answer to requests from designers and producers for information concerning the modulus of elasticity of lightweight aggregate concrete, stress-strain relations of several different designs were determined. Each of the stress-strain curves

TABLE NO. 1

Source	Cement Sacks	Aggregate - Wt. - Lbs. Coarse	Fine	Fine Sand	Total Water Gals.	W/C Ratio Gal./Sk.	Yield Cu.Ft.	Cement Factor Sk/Cu.Yd.	Slump Inches	Air %
Expanded clay, fine and coarse aggregate D-11, Houston, Texas	6	983.0	1069.0		64.61	10.77	27.00	6	3	9.5
Expanded shale - fine and coarse aggregate plus field sand, D-5, San Antonio, Texas	6	837.0	544.5	544.5	48.51	8.08	27.00	6	2 7/8	7.4
Expanded shale - fine aggregate only, D-6, San Antonio, Texas	6		1728.0		52.80	8.80	27.00	6	1 7/8	8.3
Expanded shale - fine aggregate plus admix sand, Strawn, Texas	6	1320.0		330.0	49.5	8.25	27.00	6	2 1/2	11.7



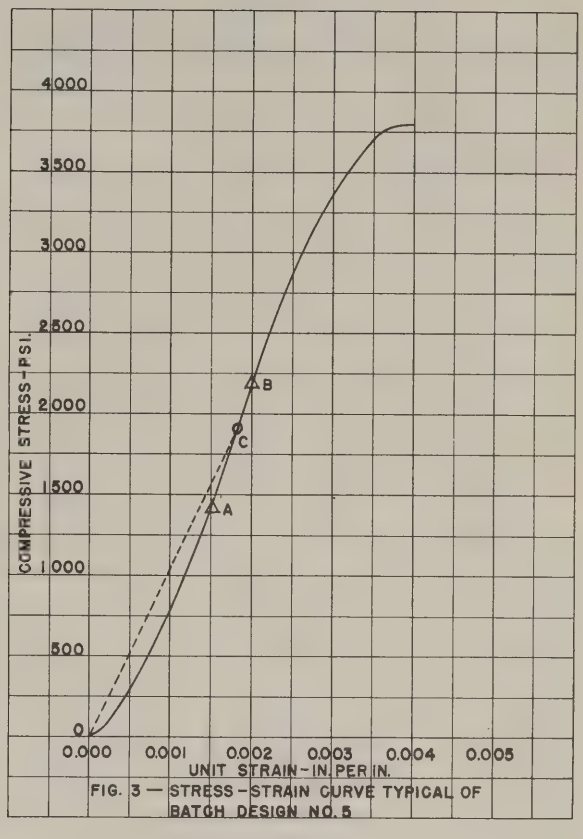


FIG. 3 — STRESS-STRAIN CURVE TYPICAL OF BATCH DESIGN NO. 5

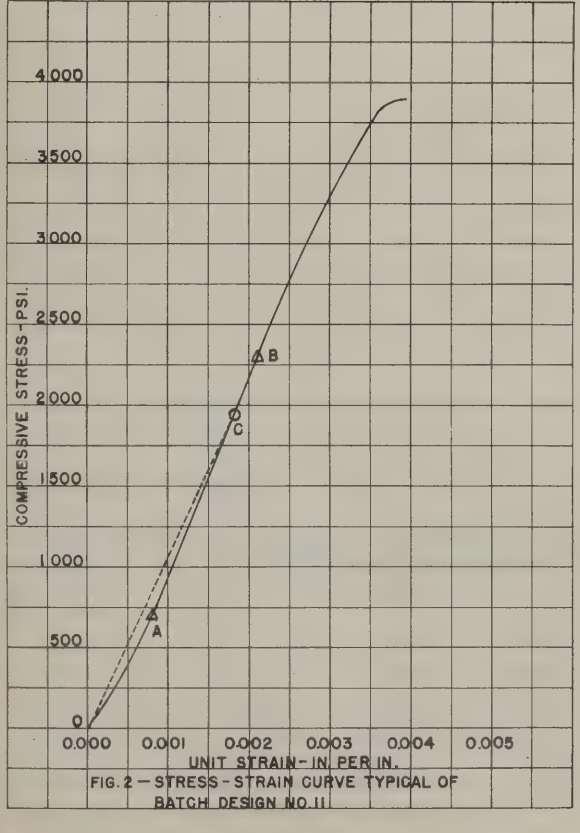


FIG. 2 — STRESS-STRAIN CURVE TYPICAL OF BATCH DESIGN NO. II

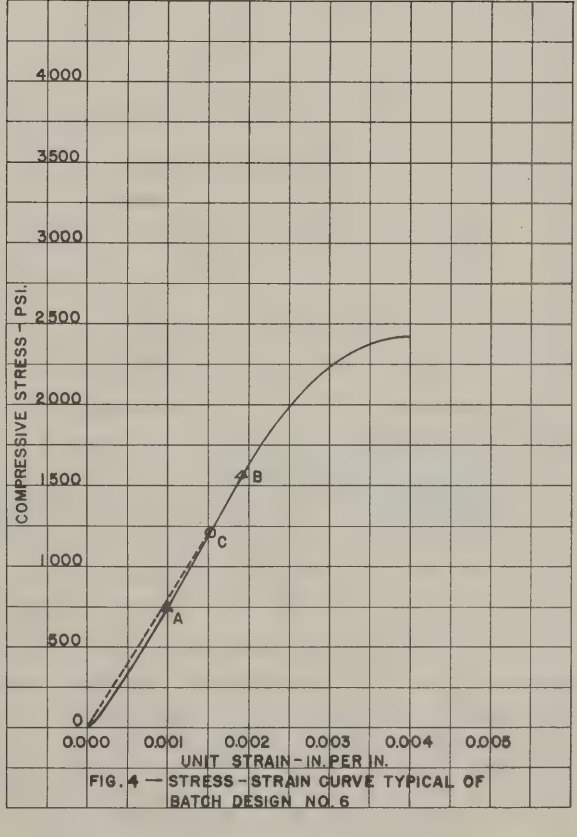


FIG. 4 — STRESS-STRAIN CURVE TYPICAL OF BATCH DESIGN NO. 6

(Figures 2, 3, and 4) is typical of a particular design (see Designs 5, 6, and 11 in Table 1).

The compression tests were performed on a 200,000-pound capacity Southwark-Emery press. The deformations were measured with a micrometer dial attached to the frame of the press by means of an angle-iron bridge. Simultaneous readings of load and deformations were recorded at intervals of 0.002 inches deformation. To eliminate irregularities due to seating phenomena, a load of 1,000 pounds was applied and released three times before data were recorded. The load was then continuously increased until failure of the specimen occurred.

Two methods of computing the modulus of elasticity were employed. One was the tangent to the curve of the point of steepest slope which usually occurred at about 50 per cent of ultimate strength. The second was the secant modulus at 50 per cent of ultimate strength. Previous experience with repeated loading and unloading of specimens having similar characteristics has indicated that the entire lower portion of the stress-strain curve probably will become a straight line parallel to the tangent which had the steepest slope on the first loading.

The stress-strain curve in Figure 2 is typical of Design No. 11 in which both fine and coarse aggregates were expanded clay from the Houston area. The portion of the Curve AB is the steepest slope. Average maximum tangent modulus for a group of three is 1,310,000 p.s.i. The dotted OC is the secant modulus at 50 per cent of ultimate strength. Its average value is 1,040,000 p.s.i.

The stress-strain curve in Figure 3 is typical of Design No. 5 in which fine and coarse aggregates were expanded shale from the San Antonio area. Local field sand was admixed with the fine aggregate. The portion of Curve AB is the steepest slope. Average maximum modulus for a group of three is 1,482,000 p.s.i. The dotted line OC is the secant modulus at 50 per cent of ultimate strength. Its average value is 1,020,000 p.s.i.

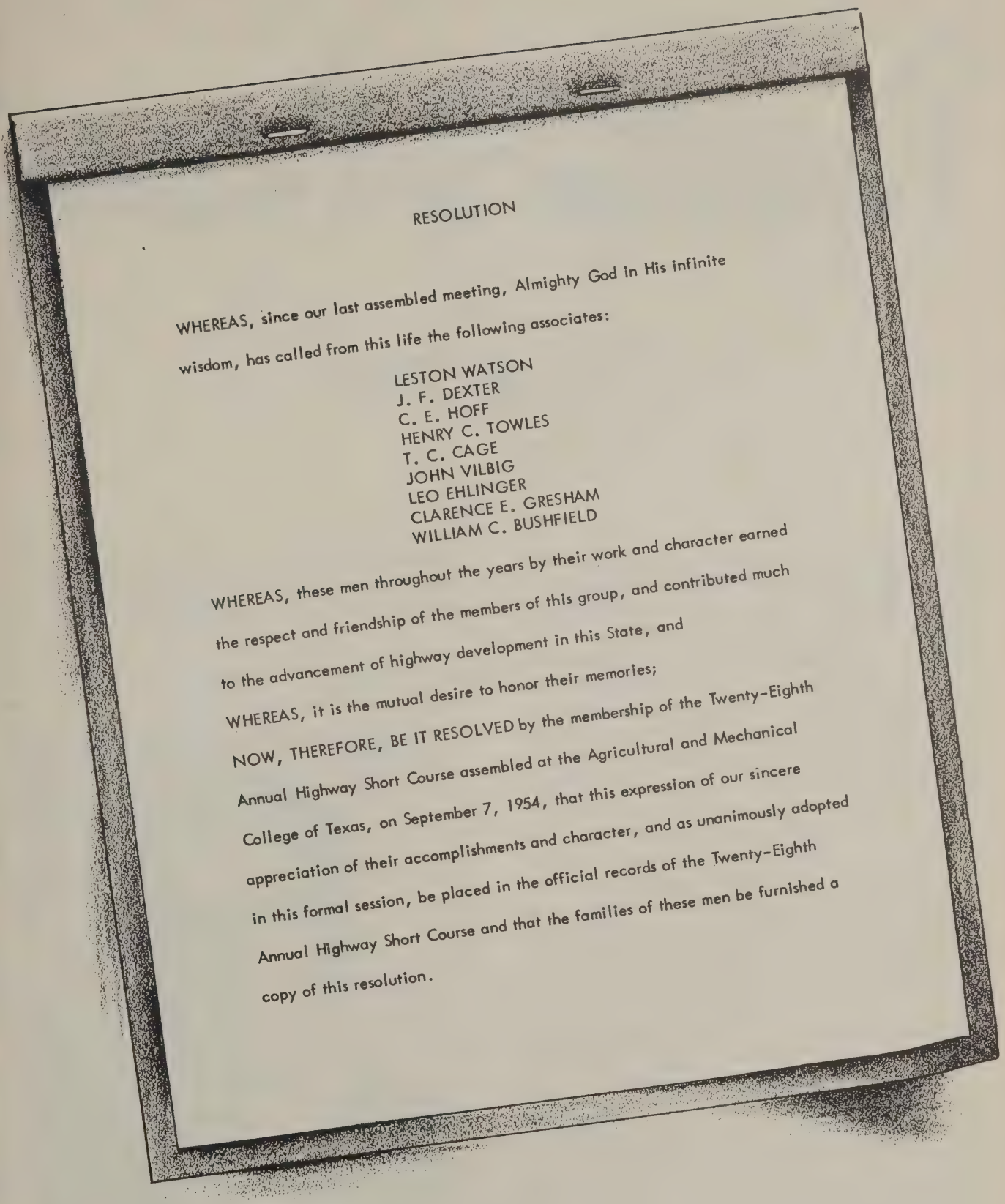
The stress-strain curve in Figure 4 is typical of Design No. 6 in which only fine aggregate (100 per cent passing the 3/8-inch round opening screen) from the San Antonio area was used. The portion of Curve AB is the steepest slope. Average maximum modulus for a group of three is 847,000 p.s.i. The dotted line OC is the secant modulus at 50 per cent of ultimate strength. Its average value is 720,000 p.s.i.

CONCLUSIONS: One definite conclusion stands out. Lightweight aggregate concrete batches cannot be designed by the usual methods which are applicable to conventional aggregates. The only satisfactory method of batch design that has been devised is based on the determination by trial batches of the total weight of combined aggregates which will yield the required quantity of finished concrete when combined with fixed quantities of cement and air-entrained admixture and the quantity of water necessary to produce the desired workability. Specific gravity in the saturated, surface-dry condition and unit weight of lightweight aggregates are elusive and indefinite properties. The design of a concrete mix should not be based on determinations as undependable as these are.

The apparent retrogression in flexural strength is not a characteristic of lightweight concrete. Dry beams break at a lower load than wet beams and the difference represents only a difference in testing procedure.

The compressive and flexural strengths of lightweight aggregate concrete containing six sacks of cement per cubic yard compare favorably with those of concrete made from conventional aggregates and containing five sacks of cement per cubic yard.

The modulus of elasticity of lightweight aggregate concrete is lower than the value usually assigned to concrete made with conventional aggregates and it varies somewhat with the particular aggregates used. Maximum modulus of elasticity ranged from 0.85×10^6 p.s.i. to 1.48×10^6 p.s.i. The secant modulus at 50 per cent of ultimate strength ranged from 0.72×10^6 p.s.i. to 1.04×10^6 p.s.i.



RESOLUTION

WHEREAS, since our last assembled meeting, Almighty God in His infinite wisdom, has called from this life the following associates:

LESTON WATSON
J. F. DEXTER
C. E. HOFF
HENRY C. TOWLES
T. C. CAGE
JOHN VILBIG
LEO EHLINGER
CLARENCE E. GRESHAM
WILLIAM C. BUSHFIELD

WHEREAS, these men throughout the years by their work and character earned the respect and friendship of the members of this group, and contributed much to the advancement of highway development in this State, and

WHEREAS, it is the mutual desire to honor their memories;

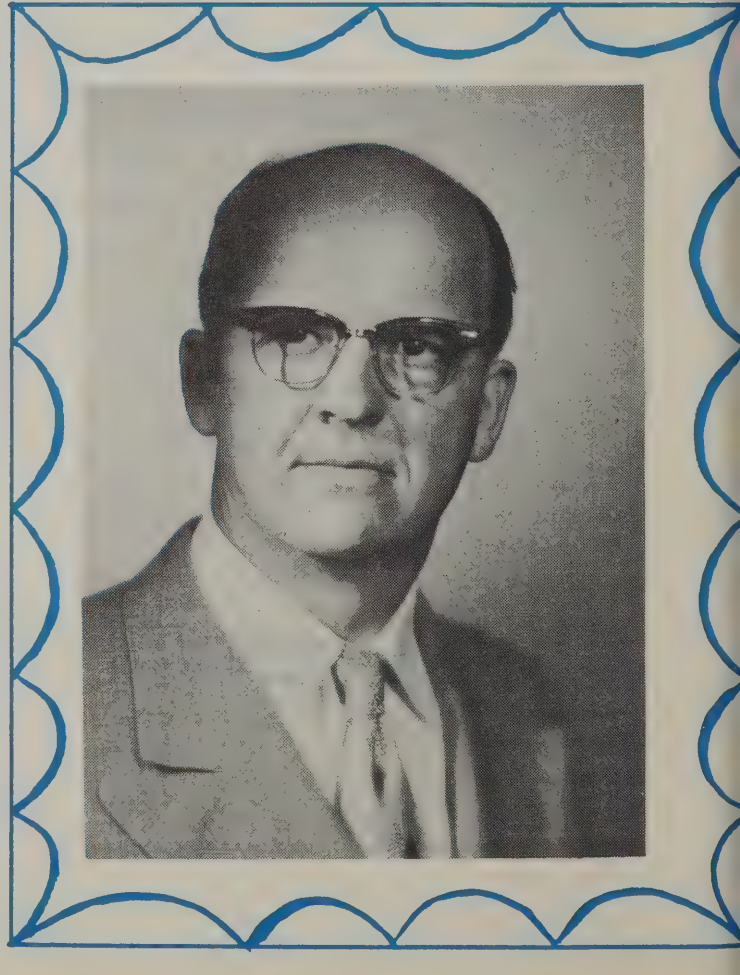
NOW, THEREFORE, BE IT RESOLVED by the membership of the Twenty-Eighth Annual Highway Short Course assembled at the Agricultural and Mechanical College of Texas, on September 7, 1954, that this expression of our sincere appreciation of their accomplishments and character, and as unanimously adopted in this formal session, be placed in the official records of the Twenty-Eighth Annual Highway Short Course and that the families of these men be furnished a copy of this resolution.

Possibilities of Precast

The subject of my discussion is "The Possibilities of Precast Concrete in Highway Structures." I want to expand this to include prestressed concrete as well as conventionally reinforced concrete.

In making a study of highway costs of now and 1925, the Bureau of Public Roads publication, "Highway Statistics," 1952, indicates that the cost of earth work has increased only 10-12 per cent. However, concrete paving is up 78 per cent and unit prices for bridge construction have increased to two and one-half times the 1925 averages. A large part of the difference could be explained by the high degree of mechanization of earth work and the relative lack of mechanization and failure to improve methods for bridge work. In an effort to find part of the answer to the problem of building low-cost bridges, the Texas Highway Department is sponsoring a research project here at A & M on the use of precast and prestressed elements in highway structures.

Any time a new development comes along, it immediately meets with strong opposition from the die-hard faction that is against any kind of change. Precasting and prestressing have met with this normal resistance and that is good. It is good because there is no way to get the engineering profession or the contractors to accept



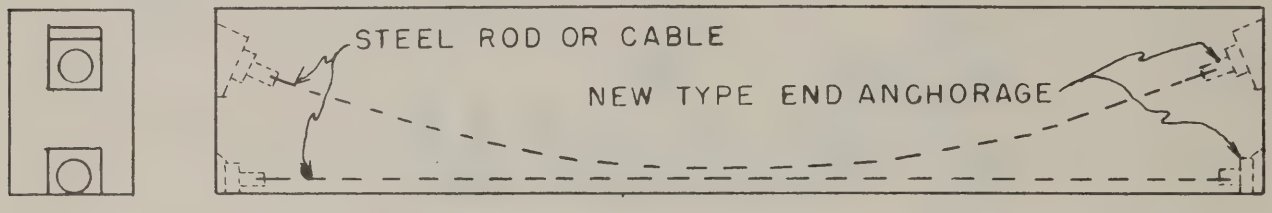
Concrete in Highway Structures

Truman R. Jones Jr.
Assistant Research Engineer
A & M College of Texas
College Station, Texas

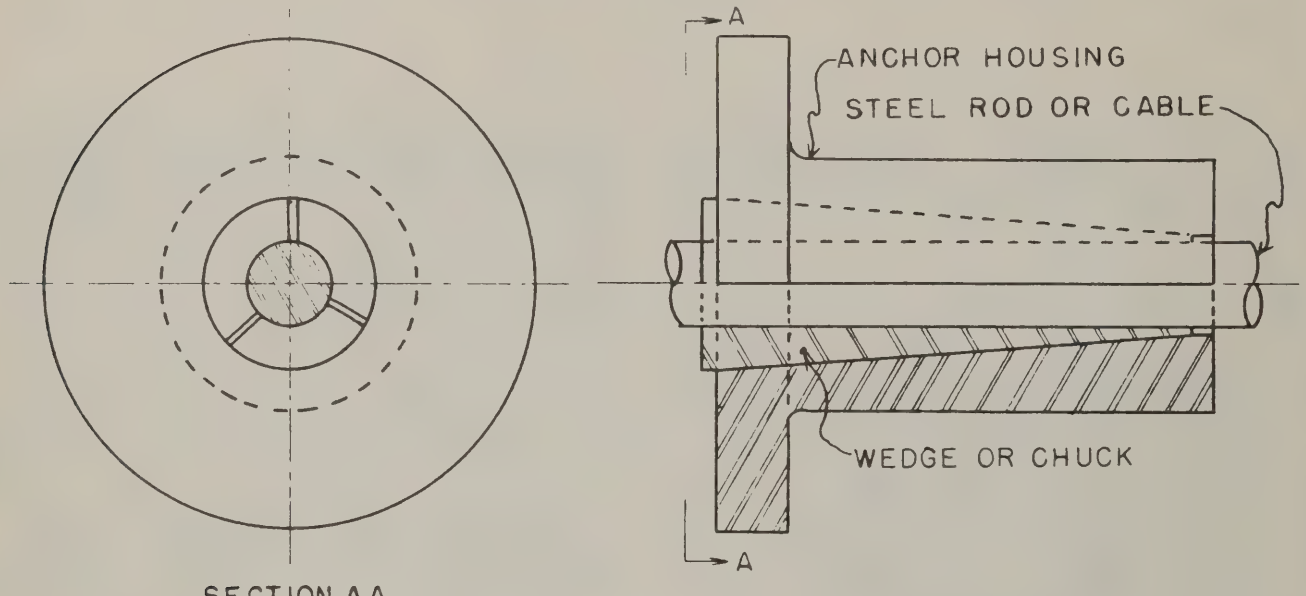
a new development in the dark, it must be tried and tested. I would say that precasting and prestressing have been investigated sufficiently to warrant their use in present-day bridge building. To demonstrate this point, I refer you to the Bible, I Kings 6:7, where we are told that King Solomon's temple was built of stones cut squared and numbered in the quarry and of timbers prepared in the forests of Lebanon and then shipped to the job site for assembly, so that no metal tools were needed or used, in erecting the building. This is one of the earlier examples of prefabricating.

There is nothing particularly mysterious about prestressing concrete members for structures. Prestressed concrete employs the same fundamental principle that has been employed for ages by barrel makers who force metal hoops tightly around wooden staves to hold them in compression, thus preventing any cracks opening up due to the bursting forces of the material placed in the barrels. Patents were issued for prestressing concrete as early as 1881 and even though the first attempts were unsuccessful, European engineers have been using it satisfactorily for the past 30 years. You may recall from Mr. Stephenson's remarks yesterday that we have only had a printed AASHO bridge specification since 1931. The relatively recent development of high-strength steel with ultimate strength values in the neighborhood of 250,000 p.s.i. makes it practical to employ the prestressing principle in concrete and various other types of construction. Prestressing, as a general term, falls into two classes, pre-tensioned and post-tensioned. In pre-tensioning, the steel is tensioned before pouring the concrete, whereas, in post-tensioning, the steel is stressed after the concrete is poured and sets up. In the case of a post-tensioned, prestressed concrete beam, the steel is placed in relatively the same position as conventionally reinforced concrete, but bond is prevented by wrapping the steel or placing it inside a thin-walled tube. Frequently, holes are formed for the steel with rubber hose or some similar

device and the steel is pulled through the hole after the concrete sets up. After the concrete has cured sufficiently the steel is stressed to as much as 0.7 the ultimate strength and anchored at the ends of the beam, transferring the same total load to the beam that exists in the steel. This causes high compressive stresses on the tension side of the beam and in the ideal design leaves a zero stress in the top fibers. When the beam is placed in service the load must be sufficiently large to reduce the compressive stress in the bottom of the beam to zero or actually to cause a small tensile stress before cracking the bottom fibers of the concrete. In fact, this practically doubles the load-carrying capacity, as compared to

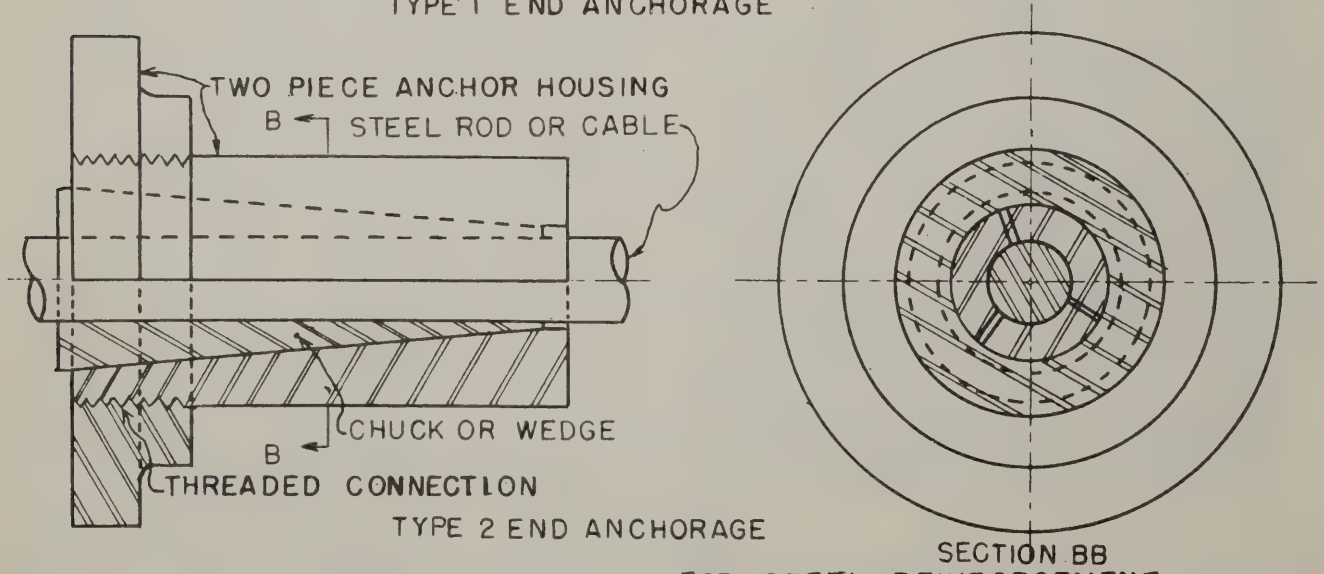


TYPICAL PRESTRESSED CONCRETE BEAM



SECTION AA

TYPE 1 END ANCHORAGE



TYPE 2 END ANCHORAGE

SECTION BB

NEW TYPE END ANCHORAGE FOR STEEL REINFORCEMENT IN PRESTRESSED CONCRETE CONSTRUCTION

conventionally reinforced concrete beams of equal size, by making all of the concrete work to carry the loads, instead of just using the portion above the neutral axis. Prestressing has many advantages. We save materials. This reduces the dead load and saves money in handling costs, freight, and erection, and even reduces the cost of the substructures. We have the ideal concrete, without cracks, and the maintenance costs are practically zero.

Precasting concrete members will make it possible to utilize all the mechanical devices now available to manufacturers of concrete products. By doing the work in centralized locations ideal laboratory control of material can be effected and mass production principles can be applied. These points have been demonstrated as facts by the building industry.

Bearing in mind the potential savings in labor, job-site forming, shoring, etc. in precasting, and potential savings in materials and maintenance costs due to prestressing, let us consider the possibilities of application in highway structures.

Some of the principles previously mentioned are in use at the present time, and as taxpayers we are fortunate that the Texas Highway Department is a progressive organization. As one example I cite the type of concrete bridge being employed by the Texas Highway Department on short spans using a pan-type concrete form that requires no shoring or wood forming between supports. This is a step in the right direction toward precast construction. Bridges are being built elsewhere in the country using precast prestressed beams with poured-in-place slabs, with shear connectors joining the slabs to the beams causing composite action for supporting the live loads. Other bridges are being built using precast prestressed beams and a deck of precast prestressed slabs. A suspension bridge over the Rio Paz between El Salvador and Guatemala, Central America, has a floor system of channel-shaped prestressed members in which the member serves the function of the deck and the stringers. M. Freysinnet designed three prestressed concrete arch bridges for a four-lane highway in Venezuela with the main spans approximately 500 feet long and a clear height up to 246 feet above stream bed in which practically the entire structure was made up of prestressed precast units. Multiple use of all forms was thereby possible, and relatively little shoring was necessary. A precast bridge approximately two miles in length has been built over the Bay St. Louis on U. S. 90 along the Mississippi Gulf Coast. The bents are made up of precast piles with poured-in-place caps. The stringers and deck are cast integrally on the shore in units providing two traffic lanes and spanning 41 feet. These large units are floated into position. There are several other outstanding accomplishments of this type that could be cited if time would permit.

Now let's consider what the future holds. Earlier we outlined the possible material and weight savings due to prestressing, we may be able to reduce this by one-third using lightweight aggregate. Dependable concrete batches using expanded shales and other lightweight aggregates, with compressive stresses up to 6,000 pounds per square inch, are now being used in the building industry. With proper curing we can attain strengths of 3,000 to 4,000 p. s. i. in 12 to 24 hours without undue cost or trouble. As soon as we establish some of the characteristics of this concrete, such as the modulus of elasticity and plastic flow, we will be able to use it in prestressed work as well as simple precasting. However, prestressed, precast concrete piles, precast abutments and bent caps, and precast elements for stringers and decks are all practical, and enough information is now available to design them safely. I would make a guess that 80 to 90 per cent of the bridges built in Texas are less than 60 feet long. For these bridges it would be possible to deliver elements in trucks to the job site where three men with one rig could erect an entire structure in very little more time than it now takes to drive the piles. There are unlimited pos-

sibilities for use in the maintenance and improvement of existing structures. A great deal of time and effort can also be saved in the widening of existing bridges. There are many instances where an entire new bridge can be built using precasting and prestressing for less money than we now spend strengthening and widening an existing structure.

Facilities are now commercially available for the manufacture of the elements described this morning in all the major cities. However, most of these plants are manufacturing members for buildings and maximum savings cannot be realized until the Highway Department is in a position to adopt standard shapes and sizes, as in the steel industry. In our research project, we have worked up information, more or less in an exploratory manner, on beams of several different sizes and shapes which could be precast very economically. At the present the best design appears to be an I-shaped member that would be placed side by side on the supports, serving as the stringers and deck at the same time. These members would be stressed laterally across the bridge after placing and would function as a concrete slab the full depth of the beam. The members are small enough for easy handling and make maximum use of all material with a minimum dead load. For instance, with a member twenty inches deep you could span up to 60 feet for an H 20 loading on a simple span. These members would weigh approximately 16,000 pounds each, and of course shorter spans with lower load requirements would decrease this accordingly. Most District Engineers could make considerable savings by setting up their own casting yards for making members to be used in their maintenance programs. When these methods have been employed for some time, it may be cheaper to buy the various members from commercial sources.

Many District Engineers have small bridges with concrete abutments, but with superstructures that need replacing. For structures of this type a farmer could drive to town in the morning over the old bridge and go home in the evening on a new concrete deck structure.

Although prestressed concrete promises great savings using the systems now available, there are still many improvements that can be made. For example, the end anchorages now in use for post-tensioned construction have three district disadvantages:

(a) They are too expensive.

(b) It is necessary to order the steel to exact lengths from the factory, which causes delays on the job and leaves practically no leeway for correcting the inevitable field errors.

(c) All of the present systems employing the use of large diameter rods and cables have shims or fittings that protrude past the bearing plate at each end of the member. This undesirable protrusion will amount to as much as three or four inches for beams 50 feet in length. This is undesirable because it necessitates wider bent caps and abutments.

As a part of our work at the Engineering Experiment Station we are developing a new end anchor, as shown in the figure, to correct these three problems. It has a low initial cost, is easy to install, can be used with steel rods or cables ordered to approximate lengths, and the entire anchor will fit inside the beams, allowing them to be placed end to end with a maximum clearance which in turn permits the use of bent caps and abutment seats of minimum widths. It will be necessary to make some physical tests of these devices before they can be put in use, but if the finished product measures up to our expectations it will save a considerable amount of money.

In summing up this discussion let me say that we do not expect precasting or prestressing concrete to be a cure-all solution to the bridge building problem but if savings of as little as 3 per cent to 5 per cent can be effected, we can save from $500,000 to $1,000,000 per year in Texas alone. The possibilities are a great deal greater. We invite suggestions and advice from each of you, and will answer any questions you may wish to ask to the best of our ability.

CURRENT DESIGN METHODS FOR DRILLED SHAFT FOUNDATIONS

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When asked to prepare a paper for presentation to this group, I began a search in an effort to determine which data concerning Current Design Methods had previously been disseminated to our organization. As a result, I found that several excellent dissertations had been published concerning both the theory and mechanics involved in determining allowable loads on drilled shafts. A paper by Mr. M. D. Shelby and Mr. Pat Clark, and one by Mr. Chester McDowell and the Materials and Tests Division at Camp Hubbard, utilizes a rigorous, mathematically exact design which culminates in an easily expressed graphical solution. The procedure followed by Mr. Fred Harris can be described as an analytical solution which is solved in part graphically and in part analytically.

I feel that a discussion by myself on the mechanics and theory alone, which has already been presented, would be superfluous. As a result, I have decided to arrange this paper in such a manner that it will detail the design features for drilled shafts from its initial stage down to and including parts of the several papers previously mentioned.

There are many factors in the selection of the proper and adequate foundation for any type of bridge or structure. However, whether the designer ultimately decides upon a drilled shaft, or another type, a careful study must be made before a decision can be reached as to which foundation is best suited for a particular situation. Naturally, we would not first make a decision to use a drilled shaft and then determine how we would fit this design into the formations encountered. So, regardless of the ultimate design, the initial study and exploration would remain the same.

In essence, the design problem may be summarized thusly. The designer first starts as far back as existing data will permit. From this point, he proceeds in a straight line and as he learns more of the features of the stream or crossing over which the proposed structure will span, preconceived ideas may possibly drop out and new ideas will take their places. Then, in the final analysis the designer is usually left with one or two possible methods of design from which a final decision must be made.

If the proposed structure is in an area which is geologically consistent, both horizontally and vertically, our problem is simplified. Conversely, if the structure is in an area of erratic geological formation, all available data is necessary. A record of high water, the magnitude and rate of runoff in the area above the structure, the amount of debris carried by the stream, evidence of scour, evidence of the stream having shifted its course or a possibility that it might do so, any recent man-made change or development above the stream which would influence runoff, and the topography are factors which will definitely influence our thinking in determining the type of foundation to be used.

Some of this data may be found in our office files. Physical evidence at the proposed crossing site if often available, and an additional source of information may be gathered by the Resident Engineer in his conversations with the older inhabitants of the area.

The first material step to be taken as an aid in intelligent design is a thorough exploration job consistent with the difficulty in crossing the proposed site and the standards expected of the new structure. There is no way of predetermining the number of borings necessary to get adequate information. However, enough borings must be taken to enable us to plot the stratigraphy throughout and beyond the length of the proposed structure. Then, when the job is placed under construction, we will know what to expect and where to expect it.

With boring data and soil tests available, we must determine to our satisfaction that the soil stratigraphy is as accurate as may be warranted in keeping with the proposed structure. From our drilling report and test data, a soil formation profile is drawn, and an outline of the new structure is superimposed thereon. We start at the natural ground line and proceed downward tying each individual formation together. This is done by comparing the visual description of the material, its soil constants, its shrinkage factor, its moisture content, its density, its strength properties as indicated by triaxial shear tests on undisturbed samples, and the penetrometer values for the material.

With this information available, we are in position to trace the underlying soil formations throughout the site, and study their possible effects upon a new bridge. Actually, this amounts to nothing more than a comprehensive soil profile. The stratigraphy should be worked out to a depth of at least twice the width of the new structure below the deepest proposed cut or below the stream bed.

Many questions will arise which must be answered, and the information determined from the profile as outlined above is an absolute necessity. Without this knowledge, one is unable to make an intelligent analysis as to the type of foundation which should be used.

The most important single factor in determining the feasibility of a drilled shaft, other than the requisite load-bearing ability of the formations below the footing, is the location of a formation in the stratigraphy of adequate depth below natural ground and adequate thickness of the formation itself. When designing for driven piling, we are interested in the characteristics of each formation from natural ground to some distance below the tip of the piling, whereas, in a drilled shaft design, we are unconcerned with the strength characteristics of the formations above our footing. Our chief concern here is a formation in which we can land our footing and the material some distance below. Regardless of the

strength of the formation in which we land our footing, it must be of the nature that enables us to enlarge the shaft. Naturally, this rules out landing a bell in a cohesionless material. We must either land above or below. If we land above a cohesionless material, our concern is that this material be as good or better than that in which we land our footing. If we land below, we must be far enough in order that water which often accompanies a cohesionless material may be sealed off during construction, thereby enabling the contractor to drill, bell the bottom, place steel, and fill with concrete. We are not interested in the material above our bell other than having a formation in which water can be sealed off. This is so because we do not consider a drilled shaft to carry any load in friction. This assumption may or may not be valid, but it is safe. At some future date we may be able to predict with confidence the load in which a drilled shaft may carry in friction. It now appears that friction depends almost entirely upon the type of material through which the shaft passes. At this time, however, our design for a drilled shaft is based on point bearing only.

We are always confronted with the problem as to the minimum depth below natural ground at which we may land our footing. As many of you know, we often find our stratigraphy inverted. By that, I mean that the stronger formations appear near the surface, whereas, the deeper we go the weaker the formations become. Incidentally, when such a formation is encountered, it is generally advisable to dispense with ideas of a drilled shaft, and go to one of the other types. This particular point will be covered later. Foundation engineering is not an exact science, as the argument over a minimum depth at which to land a bell shaft will verify. It should be kept in mind that all of these remarks are prefaced upon the assumption that the foundation material with which we are dealing is not of the hard shale or rock variety. There is usually no problem when you drill a few feet to bedrock. Our problem here is concerned with the plastic and cohesionless formations throughout the state.

Now to take up this issue again as to minimum depth - some engineers are completely satisfied to use a depth of at least twice the diameter of the bell where erosion is no problem. They are often justified, inasmuch as the next formation in which they can bell may be 50 feet lower. The writer feels that the footing for a drilled shaft should not be placed closer than 20 feet to natural ground upon the premise that at this depth the moisture content of the formation upon which we have placed our footing will remain constant. Consequently, there will be little breathing of the material upon which the loads are placed. After making an analysis of the soil stratigraphy and we discover that the formations from 20 feet to 30 feet are the type which have sufficient strength bearing properties, that these properties increase with depth, and the material is of such a nature that it may be belled - we have found the ideal. I do not mean to imply, however, that 20 feet is an absolute minimum and 30 feet is an absolute maximum. This can be decided only when all contingencies have been accounted for.

Another aspect, closely associated with determining the minimum depth at which to land a footing, is that of scour. Regardless of the shearing strength of a potential footing for a bell shaft, it is of no consequence if there is a possibility of the formation being undercut at some future date. Here again, an analysis of the soil profile often furnishes a clue to the possibility of such an event. This, incidentally, lends credence to the proponents of the argument that we should carry our footing to a greater depth. I should add here that there is always a chance that a stream will be deepened by man at some future date. Consequently, it is easier to allow for this factor in the design stage than be faced with it several years after the bridge is built.

Still another point related to the same topic of inadequate footing depth concerns the shaft design in the abutment bents of a structure. Merely

because our loads are lighter in the abutment bents compared to the interior bents, does not always justify landing our abutment footing at a more shallow depth. If the footings are landed too high, additional stresses imparted to the embankment by the shafts may cause the fill adjacent to the stream to dish, or slide out, thereby carrying the footing with it.

When a load is applied to a drilled shaft, its maximum intensity extends downward through the center of the shaft and the bell to a point of almost one-half the diameter of the footing. As a result, it is not enough that the material in which the bell footing is founded be of adequate or greater strength than necessary to sustain the load itself, it must be of sufficient thickness or must be underlaid with another formation as good or better to absorb the load imposed from above. If such is not the case, the shear strength of the foundation medium upon which we place our bell is not the controlling factor. We are faced with a new one - consolidation. Consolidation, in itself, is a complex topic, which is made so by the many variables involved. Among these are the load imposed from above, the rate at which the load is imposed, moisture content of the soil formations, the thickness of the formations below the footing, the degree of confinement, and the void ratio of the soil itself.

If each footing is founded upon exactly the same material with the same void ratio, the same load, the same degree of confinement, and the formation is of the same thickness throughout the structure and the material underlying the footing formation follows these same requisites, we can reasonably predict a settlement at a future date. As you can see, however, we seldom would be so fortunate as to have this group of factors operating in our favor simultaneously. As a result, we must take into account differential settlement. Rather than place ourselves in this position, it is common practice and good engineering to keep the applied loads within a range that is well under the yield point of the formations involved. By doing so, we can usually prevent undue settlement, differential settlement, and progressive failure.

You will notice on the enclosed calculation sheets that 60 pounds per square foot (submerged density) was used as the weight of displaced material. Also, 60 pounds per square foot was used in the factor E_c. There is ample room for controversy here inasmuch as, theoretically, the wet weight of the displaced material above the water table should be used. Our difficulty lies in determining the true water table, and making an application to the soil weight. There are two schools of thought concerning this problem. One follows the dogma that the water table should be determined as accurately as possible with the wet weight of the material being used above the table and the buoyant, or submerged, weight of the material being used below the table. The second school of thought adheres to the theory that the designer should use the submerged weight in all of his calculations. In order to determine the water table, it is common procedure to drill a hole, leave it open for 24 hours, and then measure the depth to free water below the natural ground. As a result, those that adhere to the first thought base their design upon the elevation of free water. Frankly, the determination of the true water table is extremely difficult. The method used at this time to determine the water table is inaccurate, for the reason that the height to which water will rise in the drilled hole is entirely dependent upon the pressure in the water-bearing formations below. Rather than try to outguess a set of circumstances over which we have no control now, and will have none in the future, it appears justifiable from the aspect of good engineering principles to use the submerged weight (wet weight of soil minus 62.4), even though in some instances it may be deemed conservative.

From a design standpoint, our ultimate interest is in obtaining a foundation which will support the design loads with little or no deflection at the lowest possible cost. So, we cannot say that we have made a complete analysis of the foundation

until we have made an economy comparison. This always hinges upon the number of structures in a contract, the size of the structures, the load which the structures will be required to carry, the depth to which we must carry our footing to sustain these loads, the type of formations encountered and their respective strengths, and last but not least, the ease with which the foundation may be constructed.

When the final analysis is made and it appears that a drilled shaft foundation should be used for a particular project, it is good practice to drill, bell a footing, and actually examine the entire shaft and bell by visual inspection; which the engineer should check against his logs and all previous data. This is necessary in order to assure ourselves that the particular formation in which we plan to land our footing can be enlarged and will remain relatively clean for a period of time to enable us to place steel and pour concrete. Many types of clay formations are such that they will satisfy all requirements, but when we attempt to bell them, the shafts or bell portion will not stand open. The shaft itself is no problem, inasmuch as it can be cased, but the bell portion can present a difficult problem because of the unsupported overhang in the bell. You may wonder whether this information could not be garnered from undisturbed cores. Often it can, but it hasn't been proven until we have actually drilled a shaft and tried it. We have found this to be cheap insurance. If this policy is followed, it is believed that an economy will be reflected in future bid prices by contractors.

With the previous material as background data, let's examine a typical site wherein there is a possibility of using a drilled shaft. I have superimposed a five-bent, slab-span bridge on a stream profile. From the drilling logs of three borings, the formations under the bridge site were traced and tied together as shown. I have numbered the formations from 1 through 7 inclusive for reference. This stratigraphy is good for an example; however, one would rarely find the formations as consistent in actual practice as I have diagramed here. The formations are classified according to visual inspection, the penetrometer values are noted at the depths at which they were taken, and the physical strength characteristics as determined by triaxial test data on undisturbed samples, are recorded for comparison.

Formations 4 and 7 are of such nature as to make it extremely difficult to get undisturbed samples. Often in material such as this, we are able to obtain densities and moisture contents. With these two characteristics known, if such be necessary, it is possible to closely reproduce this density and moisture content within the laboratory. Then, by using a vacuum as the lateral pressure, we can conduct triaxial tests on the material and obtain a rather accurate angle of internal friction. When this procedure is followed, a word of caution is in order. At some future date some of the overburden may be removed in Formations 2 and 3 and there could be an increase in water pressure in Formation 4, which may create a quick condition in the sand, and any bearing value that had previously been obtained would be academic. For this purpose, and generally the designer is justified in doing so, I have arbitrarily assigned values of cohesion and angles of internal friction to Formations 4 and 7. It is obvious that we would not land in Formation 1. Formations 2 and 3 have low-bearing values, and since they are underlaid by Number 4, we can eliminate them.

It appears that the firm red clay in Formation 5 is the first one to have the requisites for a foundation medium. The underlying Formations 6 and 7 are both good, and if after a strength analysis, we determine that Formation 5 is suitable, our footing should stop somewhere in this strata. Formation 4 will require casing, and as a result we would be unable to land or bell higher than elevation 45 feet. We can assume here that it will be necessary to screw our casing to a depth of about 1.5 feet into the firm red clay in order to seal off the water-bearing sand above. This would bring us down to about elevation 48 feet. Then, we must

allow room for the conventional belling tool to flare out when cutting the bell. This distance depends upon the size and type of device used. At this stage of analysis, the designer would probably be unfamiliar with the type of belling device to be used, consequently, he should probably carry the footing to a depth at which there will be no question of sealing off water and cutting the bell to the required size. You will note that elevation 42 feet was selected as the appropriate depth at which to land the footing.

A foundation engineer usually considers the heavier loads first, based on the assumption that the lighter loads will take care of themselves. With this in mind, as our calculations at elevation 42 feet are borne out for the interior bents, we can work out the abutment bents. Formations 2, 3, and 4 are still unsuitable for reasons previously mentioned. Consequently, the abutment footing will probably have to be carried to about the same depth as those in the interior bents.

We have mentioned the danger of consolidation at an earlier point in this paper. As we do not generally conduct consolidation tests, it is necessary to have some assurance that undue consolidation will not take place. From a study of the log sheets on which the soil constants are plotted, you will note that the plastic limit and moisture content in both Formations 5 and 6 are relatively close together. This is in our favor, inasmuch as it indicates that this material is already in a highly consolidated state and we would not expect undue consolidation henceforth, if our loads are maintained at a reasonable range below the yield point of the material.

The relationship between the liquid limit, moisture content, and plastic limit, as drawn on the log sheet, is an excellent point at which to make an initial investigation as to the feasibility of a drilled shaft. It is only necessary to scan the relationship portrayed on the log and the designer can quickly obtain an idea of the problems involved in a drilled shaft. If the relationship is consistent as I have indicated here, the possibility of using a drilled shaft can be kept in mind. Conversely, if the relationship is inconsistent, the formations are thin, the material has a high liquid limit, the moisture content is high, and the material appears to get worse with depth, all of which may be determined by a rapid scanning of the log sheet, the designer can immediately forget any ideas he may have had pertaining to the possibility of using a shaft.

Let's return to our example. It appears at this stage that all requisites had been met for the employment of a drilled shaft. Figure 1 is a typical stress-strain curve portraying the shear strength of the material taken from Formation 5, in which we anticipate landing our footing. Figure 2 depicts the Mohr Circles which were plotted from the stress-strain data. From this, we have determined that the material in Formation 5 has a cohesion of 4,000 pounds per square foot, and an angle of friction of 4 degrees. Table IV is a work sheet illustrating the analytical solution for the load capacity of a circular footing. Only Formation 5 was calculated, inasmuch as Formations 1 through 4 inclusive had been previously eliminated for various reasons. Now again, back to our example you will see that the allowable bearing on Formation 5 is determined to be 4.3 tons per square foot. As a rapid check of this method, we may use the graphical solution shown in Table VI on which the Mohr rupture line as determined from Figure 2 has been superimposed. Then a stress line is drawn in at an angle of 34 degrees to X-axis, intersecting the X-axis at a point of pressure which is equivalent in pounds per square inch to the hydrostatic soil pressure at the point of maximum stress. In this method, the point "Z" is 0.707 times the radius of the bell below the footing. Now a stress circle is drawn as shown, which is tangent to both the stress line and the Mohr rupture line. The hydrostatic soil pressure "U" is then substracted from the vertical unit stress and the result is divided by the appropriate influence factor, which in this case is 0.808. The resulting answer of 4.2

tons per square foot is certainly close enough for a check on the previous method.

You may wonder which of the two methods has the greater merit. Personally, I think there is a place for both. The graphic method is more rapid, whereas the analytical method is easier to check when someone else works it out, because each step is tabulated.

We could consider our problem concluded at this juncture. However, I think it is advisable to reiterate a point which was made earlier. That is, after all variables have been eliminated and we feel the drilled shaft design is feasible, it is necessary to drill a test hole on the site to substantiate our findings. If we do otherwise, we cannot say that we have an adequate drilled shaft design.

Hole # 3

Co. Harris Proj. No. 17-1 Date 2/5/54 Gnd. El. 60.0 Sta. No. 270+46

Hwy. No St. 73 Control 508-1 Name of Job Dream Bayou Bridge

Depth and Bore Type	Symbol	Lt. Pressure	Sample	Sample #	Wet Density pcf	P.I.	Remarks and Description of Stratum
0		0	X	1A	114	40	Soft, Red & Grey Clay
1		0	X	2A,B	115	35	Rather Soft, Tan , Calcareous Clay
2		0,10	X	3A,B	115		
3		0,5	X	4A,B	114	35	
4		15	X	4C	116		6/6 - 7/6 (Penetrometer)
5							
6		0,5	X	5A,B	114		
7		20	X	6 B	115		
8	X	0,10	X	7A,B	115	36	
9							Loose, Grey, H_2O Bearing Sand
10					110		2/6 - 6/6 @ 9.0'
11		0,15	X	8A	125	30	Firm Red Clay
12		5,10	X	9A,B	125		
13		0,20	X	10A	126		
14		0,5	X	11B	124	29	28/6 - 28/6
15					125		
16		5,20	X	12B	126		
17		5,10	X	13A,B	125		
18							
19		0,5	X	14A	124	30	29/6 - 31/6
20		0,10	X	15B	125		
21		0,5	X	16A,B	126	29	
22		0,10	X	17A	127	20	Very Firm, Tan & Red Silty Clay with
23		5,15	X	18A,B	126		Calcareous Nodules
24							31/ - 33/6
25							
26		0,20	X	19A	128	18	
27							
28							
29		0,5	X	20B	126		
30		5,20	X	21A,B	127	21	
31							Dense, Brown Sand ; 34/6 - 47/6
32							
33							
34							
35							TABLE I
36							
37							
38							30/6 - 50/6
39							
40							

Liquid Limit - —— -

Moisture Ct. ——

Plastic Limit - - - -

0 10 20 30 40 50 60 70 80 90 100

Non Plastic

" "

Non Plastic

" "

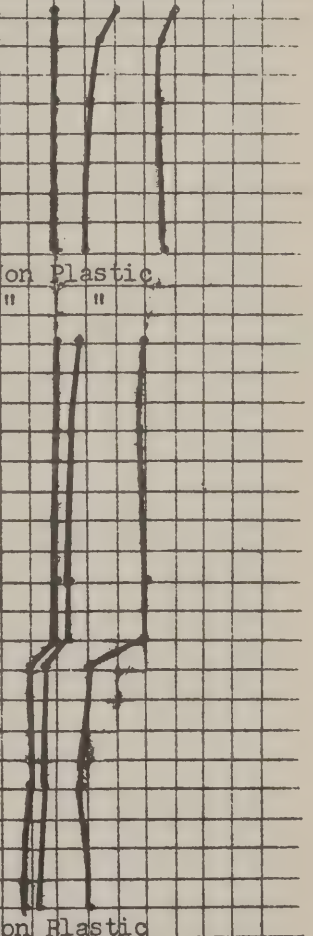
" "

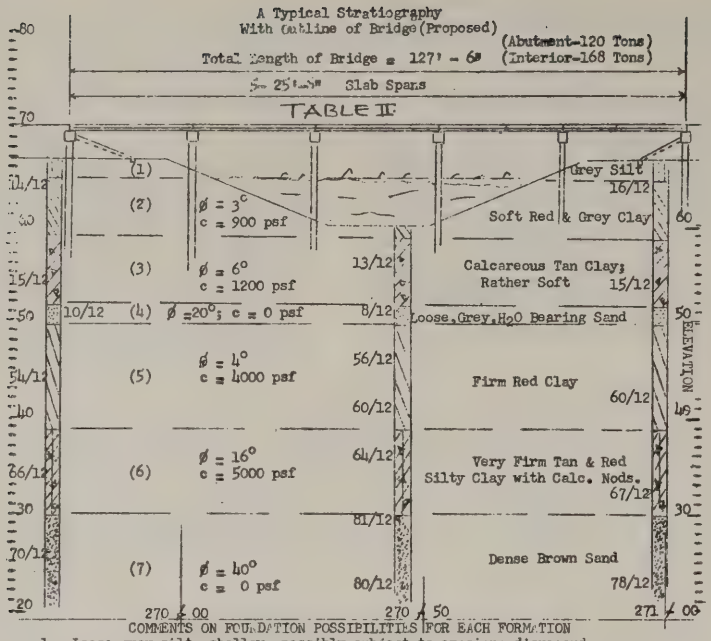
" "

" "

" "

" "





COMMENTS ON FOUNDATION POSSIBILITIES FOR EACH FORMATION

1. Loose gray silt, shallow, possibly subject to erosion; disregard.
2. Soft red and gray clay, low bearing value, possibly subject to sliding and erosion; of little value. - Disregard.
3. Rather soft calcerous tan clay, no bearing value, possibly subject to erosion. If this formation were of somewhat higher bearing value, it would probably still be ruled out inasmuch as it is underlain by a loose water bearing sand.
4. Loose gray water bearing sand, a poor formation upon which to rest a footing, and a shaft must be cased through it.
5. Firm red clay, first formation available which we might use as a footing for a drilled shaft. We would probably be unable to land higher than elevation 45.0' due to the overburden of loose gray water bearing sand, inasmuch as this sand must be sealed off in order to permit us to bell in this formation. It appears that elevation 42.0' would be the appropriate depth at which to land.
6. Formation #6 is probably an excellent material in which to bell, the bearing value is more than adequate, it underlies the firm red clay, if we wish to bell in Formation #5, and it overlies a dense brown sand.
7. The dense brown sand in Formation #7 enables us to land in formation #6 and serves as an extremely good underlying medium upon which to place a bell.

NOTE: 1. Penetrometer values are shown as recorded, for example: 13/12 indicates 13 blows at 12 inch penetration.
2. The values in Formation #4 and Formation #7 are assumed.

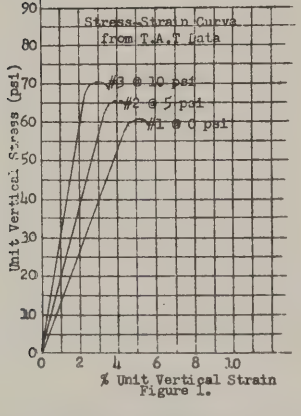


Figure 1.

TABLE III

Mohr Circles plotted from
Stress-Strain Data at Left.

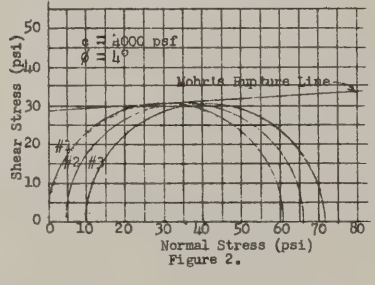


Figure 2.

The limitations and concepts of both the Graphical solution and the Analytical solution are as follows:

1. Both are based upon a Poisson's ratio of 0.5.(That is, ½ the ratio of the developed vertical stress to the applied lateral stress. Poisson's ratio may be determined rather closely by reading the vertical pressure developed when lateral pressure is applied without elongation of the specimen.)
2. The Graphical method is based upon the solution that the point of maximum shear occurs at a point 0.707 x the radius of footing on a line passing through the center of the footing for a uniform material.
3. The Analytical method indicates that the point of maximum shear occurs at a point 0.4 x the diameter of the footing. The other factors are same as #2.
4. The soil overburden is assumed to act hydrostatically in both methods. That is, the same lateral pressure is exerted as would be exertedby a liquid of equal density.
5. The depth of footing below ground surface is greater than footing diameter for both methods.
6. The assumption that the soil is a plastic or semi-plastic material.
7. The foundation soil is of uniform and/or increasing strength for a considerable distance below the footing for both methods.
8. The length of the footing (where a rectangular type is used) must not be more than 1½ times the width. Where this ratio is exceeded, a special analysis must be made. This applies to both methods.
9. Due allowance must be made for anticipated scour in computing the values of d & u.
10. As a rule of thumb, the Graphical method should not be used when the rupture line from the triaxial data is more than 26°. Also, the test results should be questioned when the angle of friction is determined to be more than 15° for a plastic material.
11. Some localized shear of high intensity will be developed near the footing edge but this is not considered serious since this thin ring of overstress soil may borrow strength from the surrounding material.
12. An appropriate safety factor is used to keep the design stresses in the soil at a point well below the yield point of the material.

Name of Job Dream Bayou Bridge : Control 508
Location State Highway 73 - Harris County : Section 1

ANALYTICAL SOLUTION FOR LOAD CAPACITY OF CIRCULAR FOOTINGS

TABLE IV

	Strata Depth & Identification	c psf	h ft.	ø	F ø	Eo psf	Ec psf	c + (Ec . tan ø) psf	P #/ft² tsf(allowable)	
0'- 2'	Loose gray silty topsoil	144		10						(1)
2'- 8'	Soft mottled red & gray clay.	900		3						(2)
8'-15'	Rather soft calc. tan clay.	1200		6						(3)
15'-17'	Loose gray water bearing sand.	0*		20*						(4)
17'-28'	Firm red clay.	4000		4	.26	1500	1570	4110	4.3	(5)
28'-37'	Very firm tan & red silty clay with calc. nodules.	5000		16						(6)
37'-51'	Dense brown sand.	0*		40*						(7)

SF - 2.0
Bent loads include weight of shafts.

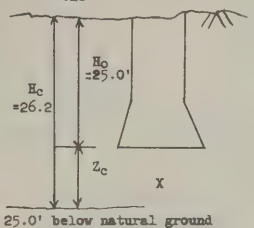
$P = \frac{c + Ec \tan \phi}{F \phi} + Eo = \frac{4000 + (1570 \times .07)}{.26} + 1500$

= 17,300 psf, ultimate or 8,650 psf allowable or; 4.3 tsf allowable.

Letting w = unit weight of soil: Eo = wHo & Ec = wHc

Fø taken from lower chart of Table V.
Z_c - 0.4 - Diameter of Bell

The unit weight "w" is calculated as wet; dry; submerged. (60pcf)
Area of 3.0' bell - 7.1 SF
Diameter of bell = 3 ft.
Z_c or point X = 1.2 ft. below bell.

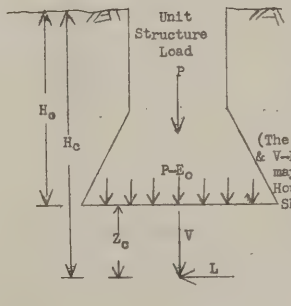


Remarks: 1. Assume the bell to land at elevation 42.0' or 25.0' below natural ground in formation #5.

2. Interior bents -(7.1 SF)(4.3 tsf) - 30.6 tons/footing-so; 168 tons/30.6 = 5.49; use 6 - 3.0' bell shafts on interior bents.

3. Abutment bents - 120 tons/30.6 - 3.92; use 4 - 3.0' bell shafts on abutments.

4. * - Assumed values.



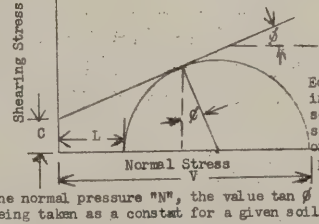
Let "w" = unit wt. of soil (Designer will have to determine whether to use "w" as Wet, Dry, or Submerged weight to fit circumstances);
$E_o = wH_o$ & $E_c = wH_c$;
Z_c = 0.4 x Diameter of Bell.(This is the point of Maximun Stress below footing.;
At Critical Depth, H_c,

$V = 0.74(P - E_o) + E_c$ - - - - - - (1)

$V - L = 0.583(P - E_o)$ - - - - - - (2)

(The values of 0.74 & 0.583 are Influence Values for V & V-L plotted against Depth for a Circular Footing, as may be found in Fig. 2, p. 76-"Use of T.A.T. on Houston Urban Expressways"; Procedure of 23rd Annual Short Course in Hwy Engi Tex. Eng. Exp. Sta; 8/1949. Poisson's ratio assumed equal to 0.5. Maximum shear stress equals approximately one-half principal stress difference.

TABLE V



From Mohr's Diagram:

$S = \frac{1}{2}(V - L)\cos\phi$ - - - - (3)

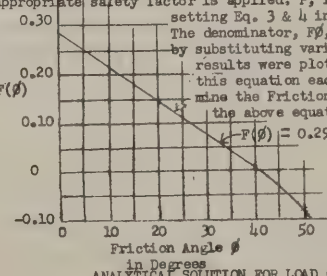
$N = V - \frac{1}{2}(V - L) - \frac{1}{2}(V - L)\sin\phi$ (4)

Then setting; $S = C + N \tan\phi$ - - - (5)

Eq. No. 5 is the basic equation for determining the physical strength properties of a soil. The cohesion "c" is the shearing strength of the soil at zero normal pressure on the plane of failure. The frictional componant of the shearing strength of a soil is defined as being directly proportional to the normal pressure "N", the value tan ø , where ø is the angle of internal friction, being taken as a constant for a given soil.

Now, solving for P; - - - - - - - - - - $P = \frac{C + E_c \tan\phi}{F(\phi)} + E_o$

P, the Ultimate Load Bearing capacity or Ultimate Stress of the formation with which we are concerned is expressed in tsf, psf, or psi, as the case may be.To this, an appropriate safety factor is applied. P, in the above equation was determined by setting Eq. 3 & 4 in Eq. 5 and solving in terms of Eq. 1 & 2. The denominator, Fø, as shown at lower left, was determined by substituting various values for a Friction Angle and the results were plotted against Fø. Instead of having to solve this equation each time, all that is required is to determine the Friction Angle, read the Fø, and substitute in the above equation.



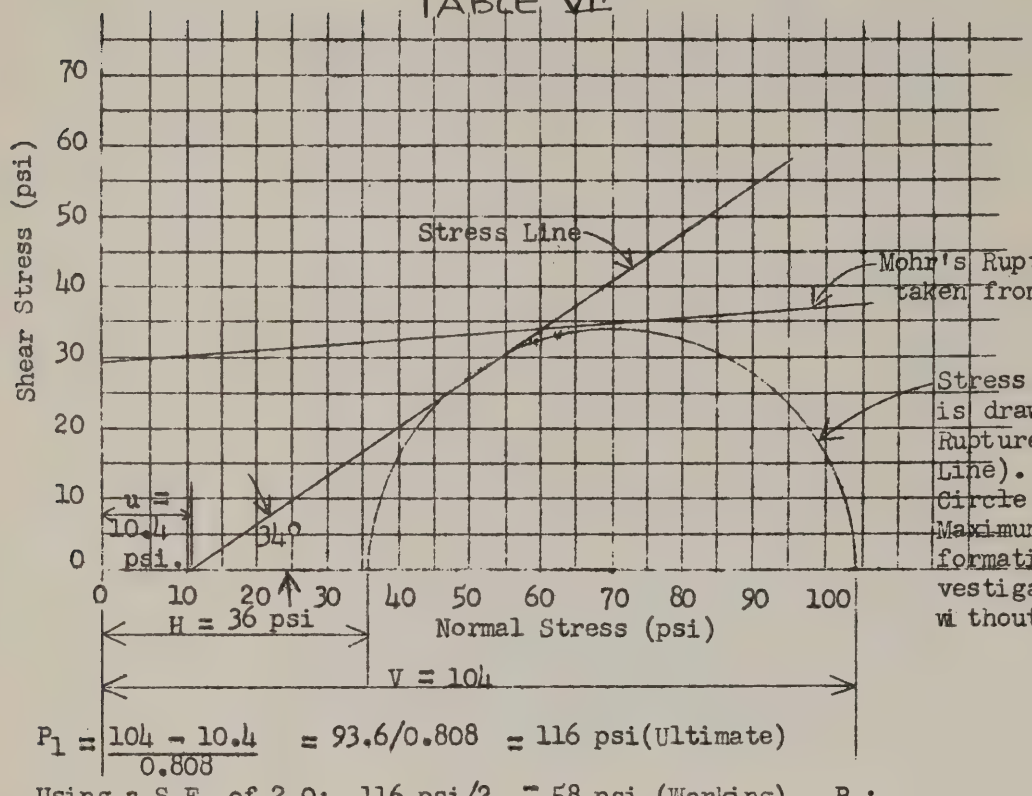
$F(\phi) = 0.2915\cos\phi - 0.4485\tan\phi + 0.2915\sin\phi\tan\phi$

Note: It should be borne in mind that this is not an exact solution inasmuch as the Influence Values were taken from a plotted curve. (eg. 0.74 & 0.583 in Eq. 1 & 2.

ANALYTICAL SOLUTION FOR LOAD CAPACITY OF CIRCULAR FOOTINGS

GRAPHICAL SOLUTION FOR DETERMINING ALLOWABLE LOADS
FOR CIRCULAR FOOTINGS

TABLE VI


Shear Stress (psi)
70
60
50
40
30
20
10
0
Stress Line
Mohr's Rupture Line as taken from Fig. 2.
Stress Circle(This is drawn tangent to Rupture Line & Stress Line). The Stress Circle represents the Maximum Stress the formation under investigation may carry without failure.
u = 10.4 psi.
34°
0 10 20 30 40 50 60 70 80 90 100
Normal Stress (psi)
H = 36 psi
V = 104

$P_1 = \frac{104 - 10.4}{0.808} = 93.6/0.808 = 116$ psi(Ultimate)

Using a S.F. of 2.0; 116 psi/2 = 58 psi (Working) = P ;

or; $P = \frac{58 \times 144}{2000} = 4.2$ tsf.

With a S.F. of 2.0, we cannot allow a load of greater than 4.2 tsf on this formation.

Symbols & Definitions:
"d" is depth in ft. from surface of ground to bottom of footing; "r" is the radius for circular footings, or radius of equivalent circular area for non-circular footings; "z" is depth in ft. from bottom of footing to point of max. shear stress in soil; "w" is density of the soil in appropriate units; "u" is the hydrostatic soil pressure based upon assumption that the soil acts as a liquid; "H" is the lateral unit stress(Minor principal stress or Sigma III); "V" is the vertical unit stress or vertical unit pressure (Major principal stress or Sigma I); "P_1" is the ultimate bearing strength of the soil; "P" is the safe unit bearing strength of the soil;

Basic Equations:
$z = 0.707\ r$; $u = w(d + z)$; $P_1 = \frac{V - u}{0.808}$ or $\frac{H - u}{0.230}$

Note: The above information may be found in a more detailed manner in "Triaxial Testing: It's Adaptation and Application to Highway Materials" Part III as developed and reported by the Soils Section of the T.H.D. Materials & Tests Laboratory.
Note 2; The Stress Line in the above diagram is drawn in at 34° because the factor of safety becomes infinite when the angle of internal friction exceeds that value and the values of 0.808 & 0.230(Influence Factors) are used as shown. Also, the Stress Line intersects the X-Axis at the value of "u" which is the Hydrostatic Soil Pressure at the point of Maximum Stress.

RESULTS OF LOAD STUDIES ON DRILLED SHAFTS*

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A & M College of Texas
College Station, Texas

No special introduction should be needed for drilled-shaft footings to Texas engineers. These footings have been the subject of a sufficient number of papers and talks and have been used on enough jobs so that most construction men are familiar with the methods and equipment employed by drilling contractors in placing them. Some of the currently used equipment for the drilling and underreaming operation of these cast-in-place piles, as well as the now routine procedure of placing the reinforcing steel and pouring the concrete, was described in the July issue of Civil Engineering magazine by Mr. Randle Alexander in his article on the Leon River Bridge.[1]

In construction work today the bored-type pile can frequently be installed more cheaply than driven piles. Hence we find the drilled shafts, with and without underreams, being used as substructures for both

*Based on research conducted by the Texas Engineering Experiment Station under a memorandum agreement between the Texas Highway Department and the Texas A & M College System.

office and industrial buildings, for highway bridge and grade separation work, and sometimes for homes.

When formations such as the hard blue shale at the Leon River Bridge site are encountered, the design of the drilled-shaft footing becomes a simple P/S = A problem with the footing load, P, being divided by the design bearing capacity, S, of the foundation material to give the required footing area, A. However, the drilled-shaft footings frequently do not extend to a dense supporting material and the proposition that the load is carried by a combination of bearing and skin friction must be considered. This complicates the design and the engineer is immediately confronted with seemingly plausible arguments – both pro and con – on some of the following issues: (1) cast-in-place versus driven piles; (2) the relative importance of skin friction and bearing in supporting pile loads; (3) the fluctuations in skin friction over extended time periods; and (4) whether a straight shaft can be extended a few additional feet to replace the benefits gained by underreaming.

Such questions as these were no doubt instrumental in prompting the Texas Highway Department to approve a study of drilled-shaft footings as a cooperative project between it and the Texas Engineering Experiment Station. This project has been active for the past twelve months.

The principal objectives of this project for the past year were to: (1) survey literature on the subject of drilled-shaft footings; (2) conduct a field exploration and soil test program to establish soil conditions at a site on the A & M College of Texas campus; (3) conduct field loading tests on drilled shafts of various sizes; and (4) develop a device for measuring bearing pressure at the base of a cast-in-place pile.

Literature Survey: Most of the available literature on the subject of cast-in-place piles represents research and observations made during the past decade. Some of the more important work reported has been performed by Meyerhof, Golder, Rodin, and Tomlinson of England, and da Costa Nunes and Vargas of Brazil. Other research has been reported by

European engineers, but much of this literature is not available in translated form.

Meyerhof developed a semiempirical equation based on the original Terzaghi equation for shallow footings.[2] Actual field load tests on ten-, twelve-, and fourteen-inch diameter piles with lengths varying from six to thirty feet gave results that agreed with calculated values.

Golder tested cast-in-place piles having diameters from sixteen to twenty-five inches and depths up to 45 feet.[3] He stated that the most reliable method of estimating the ultimate bearing capacity of a pile is by means of a loading test to failure on the pile itself. Failure was defined as the load at which there was continued penetration of the pile under no increase in load. Golder's tests gave some evidence that the allowable shear strength to be used in calculating the skin friction on a bored pile is about one-half the unconfined compression strength.

Rodin and Tomlinson studied both cast-in-place and driven piles with special tests being made to evaluate separately skin friction and base resistance for the bored piles.[4] Their tests showed that in the case of drilled piles, the base resistance was in good agreement with the theoretical ultimate bearing capacity of nine times the undisturbed shearing strength of the soil. The observed skin friction was one to two times greater than the theoretical maximum skin friction based on the shear strength of the fully softened clay.

In Brazil, da Costa Nunes and Vargas compared field load test results with values calculated by the Meyerhof equation.[5] They obtained good agreement between field loading test results and computed values for ten-inch diameter cast-in-place piles having lengths varying from five to thirty feet.

Indications are that with the increasing popularity of drilled piles, much additional information will be available in a few years.

Soil Exploration and Testing Program: The soil exploration program was conducted by the Bridge Division of the Texas Highway Department using both their equipment and personnel. At a test site on the A & M College campus four core borings were made to depths of 31, 26, 44, and 26.5 feet. Continuous undisturbed samples were taken using either a four-inch diameter Shelby tube or the Dennison core barrel sampler. The Texas Highway Department's Standard Penetration Test was also performed adjacent to core borings 2 and 4.

The undisturbed samples were cut in eight-inch lengths, weighed for density determinations, and then subjected to the confined compression test. These tests were performed using the "Trans-matic" Triaxial Compression Test Cell (designed by Fred Harris). Lateral confining pressures of 0, 5, 10, and 20 p.s.i. were

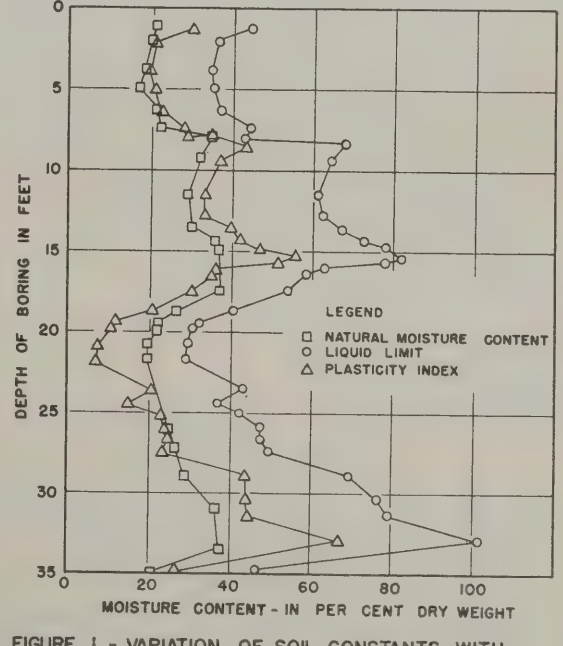


FIGURE 1 - VARIATION OF SOIL CONSTANTS WITH RESPECT TO DEPTH OF BORING 3.

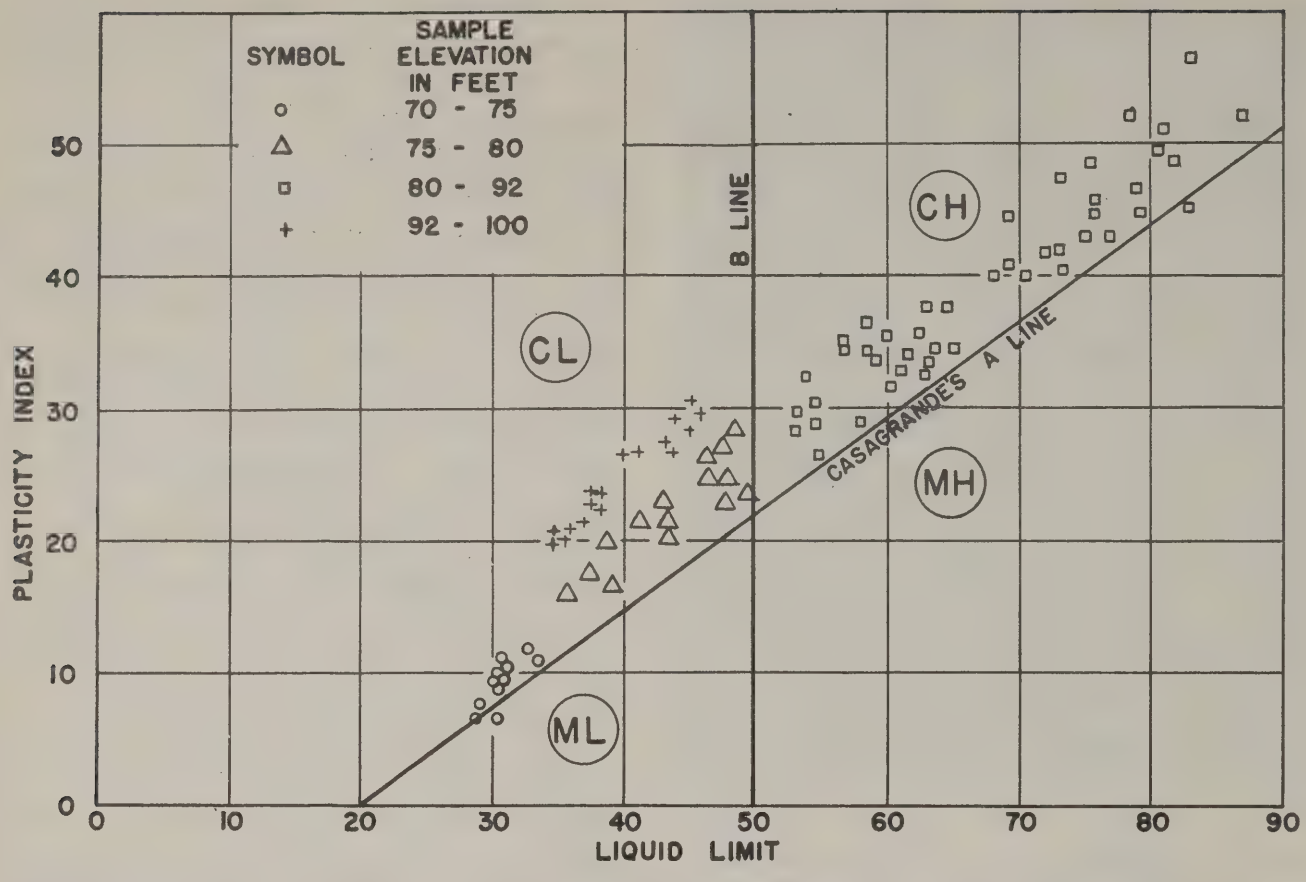


FIGURE 2 - RELATIONSHIP BETWEEN LIQUID LIMIT AND PLASTICITY INDEX FOR VARIOUS DEPTH RANGES.

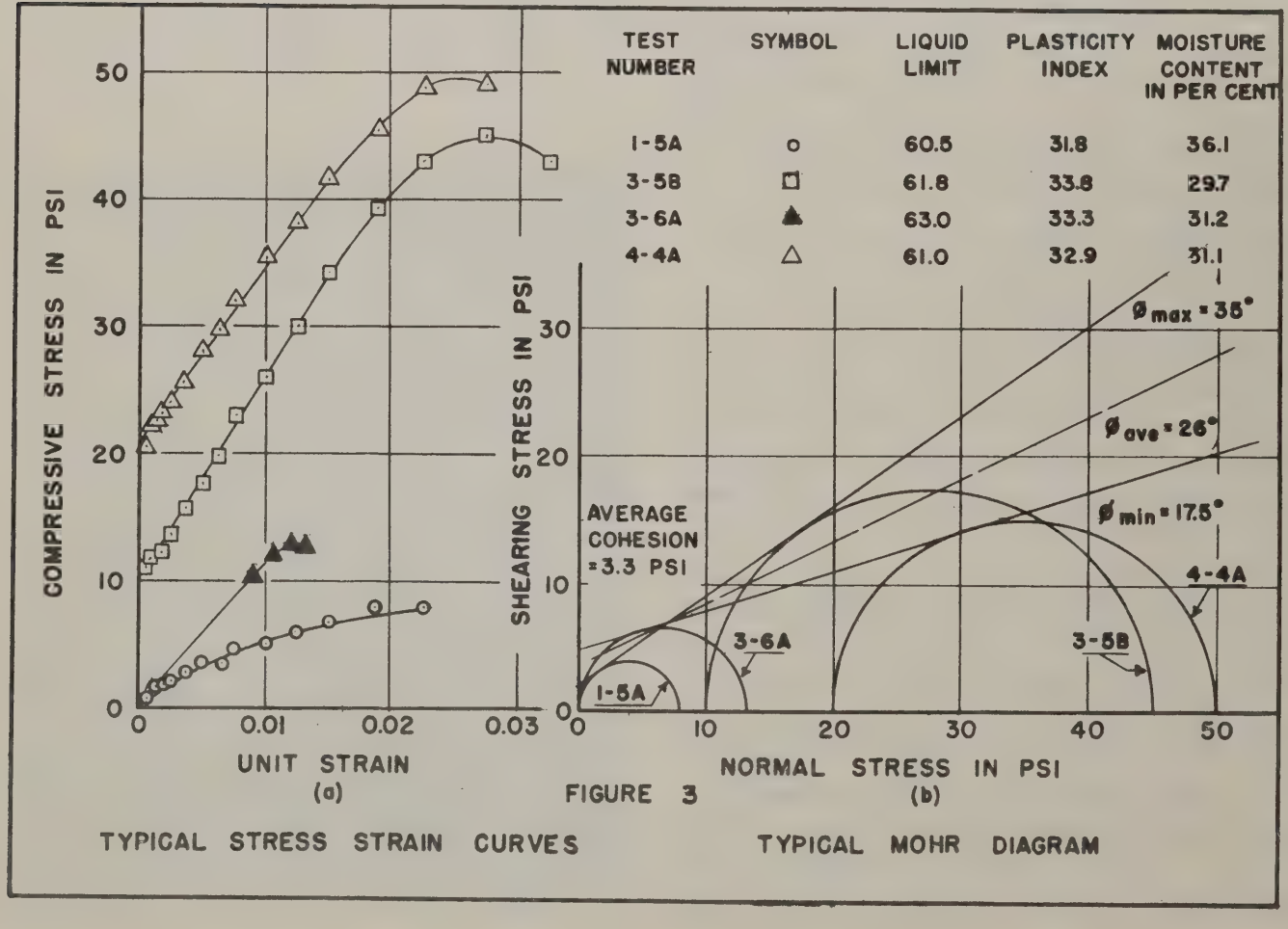


FIGURE 3

TYPICAL STRESS STRAIN CURVES

TYPICAL MOHR DIAGRAM

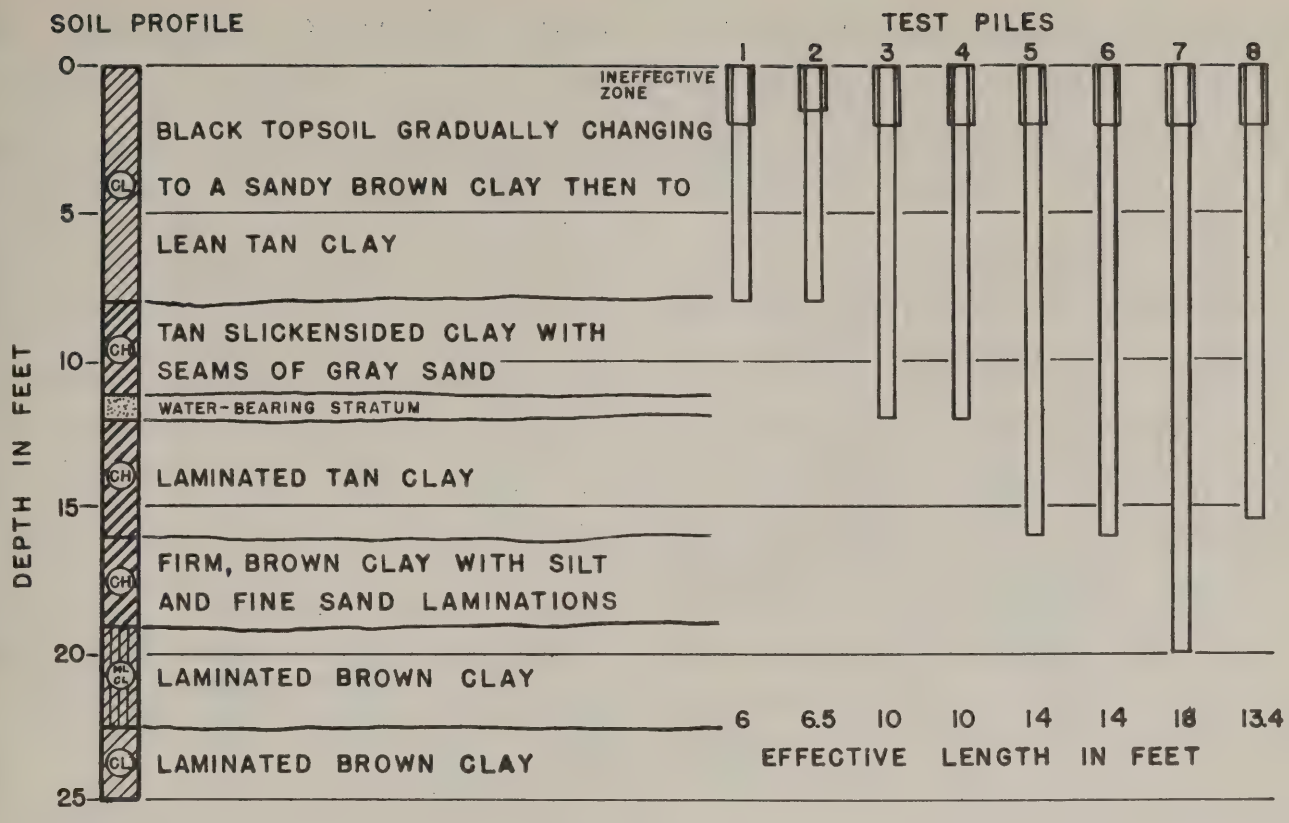


FIGURE 4

REPRESENTATIVE SOIL PROFILE AND EFFECTIVE PILE LENGTHS

Figure 5 - Observing settlement of a test pile.

used for specimens from borings 1, 2, 3, and 4, respectively. These data were later used in calculating load capacities of the test piles based on several of the currently recognized equations.

After completion of the confined compression test, the specimen was taken to the laboratory for water content, liquid limit, and plastic limit determinations. In an effort to keep technician errors as constant as possible, all plasticity determinations were made by the same operator using the same equipment. The variations of water content, liquid limit, and plastic limit with respect to depth are shown in Figure 1. Other data collected in the College Station area have shown a similar agreement between the three soil constants.

A plot of liquid limits and plasticity indices on the Casagrande plasticity chart (Figure 2) classified most of the specimens as inorganic clays of low potential compressibility, CL, and inorganic clays of high potential compressibility, CH. The grouping of data from certain depth ranges into different areas on the plasticity chart proved helpful in later interpretation of triaxial shear data for Mohr's diagrams.

Using all of the previously collected information on water content, plasticity characteristics, density, and soil profile, specimens were grouped according to stratum for drawing Mohr's diagrams. A typical set of stress-strain curves and Mohr circles are shown in Figure 3. The procedure was to calculate a maximum and minimum value for the angle of internal friction and cohension and then use an average value in calculations of pile capacities.

Field Tests: The first field tests planned were seven-inch diameter hand-augered straight shafts with no underream. Two tests were planned to each of the following effective depths: 6, 10, 14, and 18 feet. The actual drilled depths were two feet greater than the values given above. By using a two-foot section of six-inch diameter Sonotube as a form, the top two feet of each pile were rendered ineffective from the standpoint of skin friction. An average soil profile and the effective shaft lengths are shown in Figure 4. Each pile was reinforced with four one-half-inch steel bars extending the length of the pile. All concrete for the seven-inch diameter piles was hand-mixed using about a 1:2:3 mix by weight and a five-inch slump. Test cylinders indicated a 3,000 p. s. i. strength at 28 days. Load tests were conducted between five and six weeks after the piles had been placed.

The loading tests were performed by jacking against a WF beam secured to the anchor piles by a plate and strap arrangement. Loads were applied with a hydraulic jack and the deformation was measured by four dial gauges. There was some eccentricity in loading these small piles and the data from four gauges proved useful in calculating an average settlement value.

The test procedure followed that as prescribed by the Bridge Division of the Texas Highway Department as closely as possible. Load and extensometer observations were made at fifteen-minute intervals and loadings were not increased until the total additional settlement for a four-hour period had not exceeded 0.005 inch. The ultimate capacity of the pile was considered to be the load under which the total net settlement had not exceeded one-fourth inch and the settlement during the last 24 hours under this particular load had not exceeded 0.005 inch. Most of the tests were conducted for at least a 100-hour period.

The data was plotted to give both load-settlement and time-load-settlement curves. Typical data for Test 3

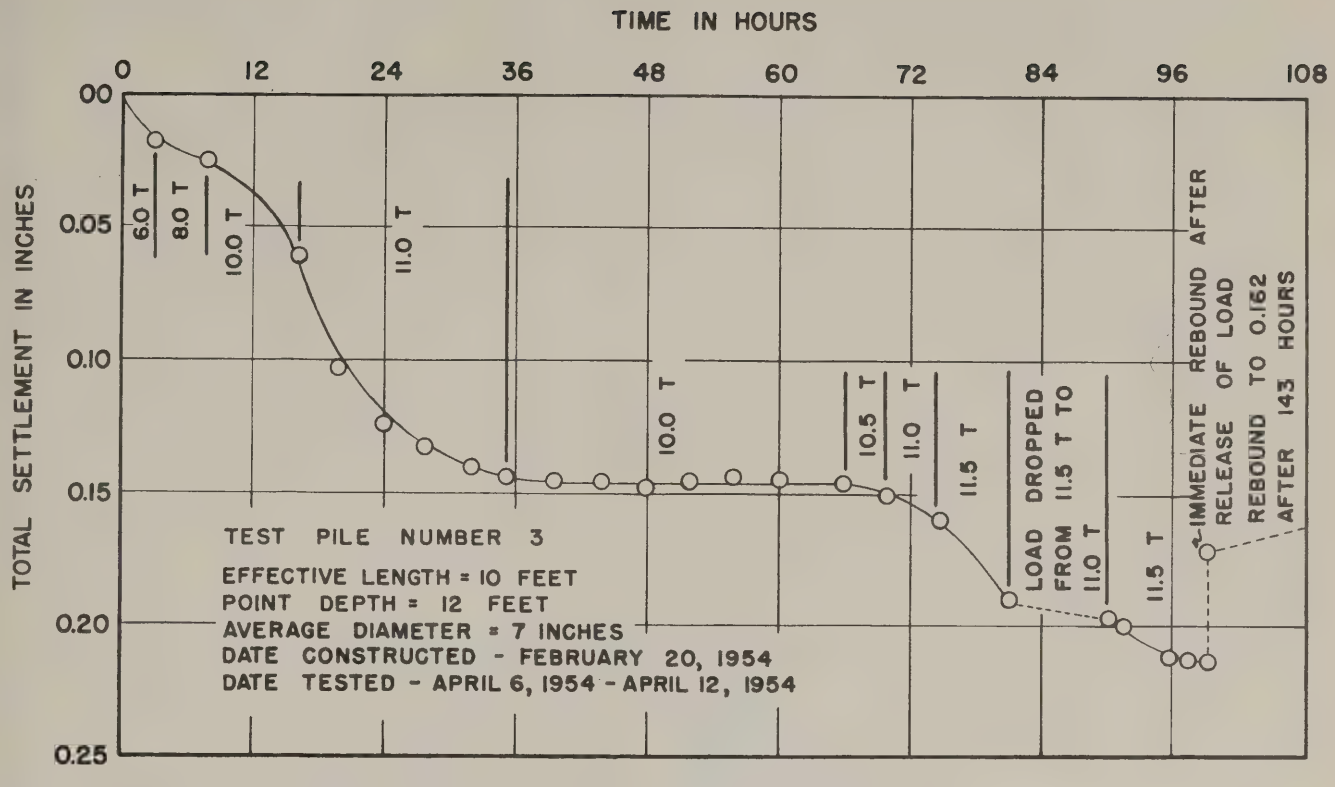


FIGURE 6 - TIME - SETTLEMENT - LOAD CURVE FOR TEST PILE NUMBER 3.

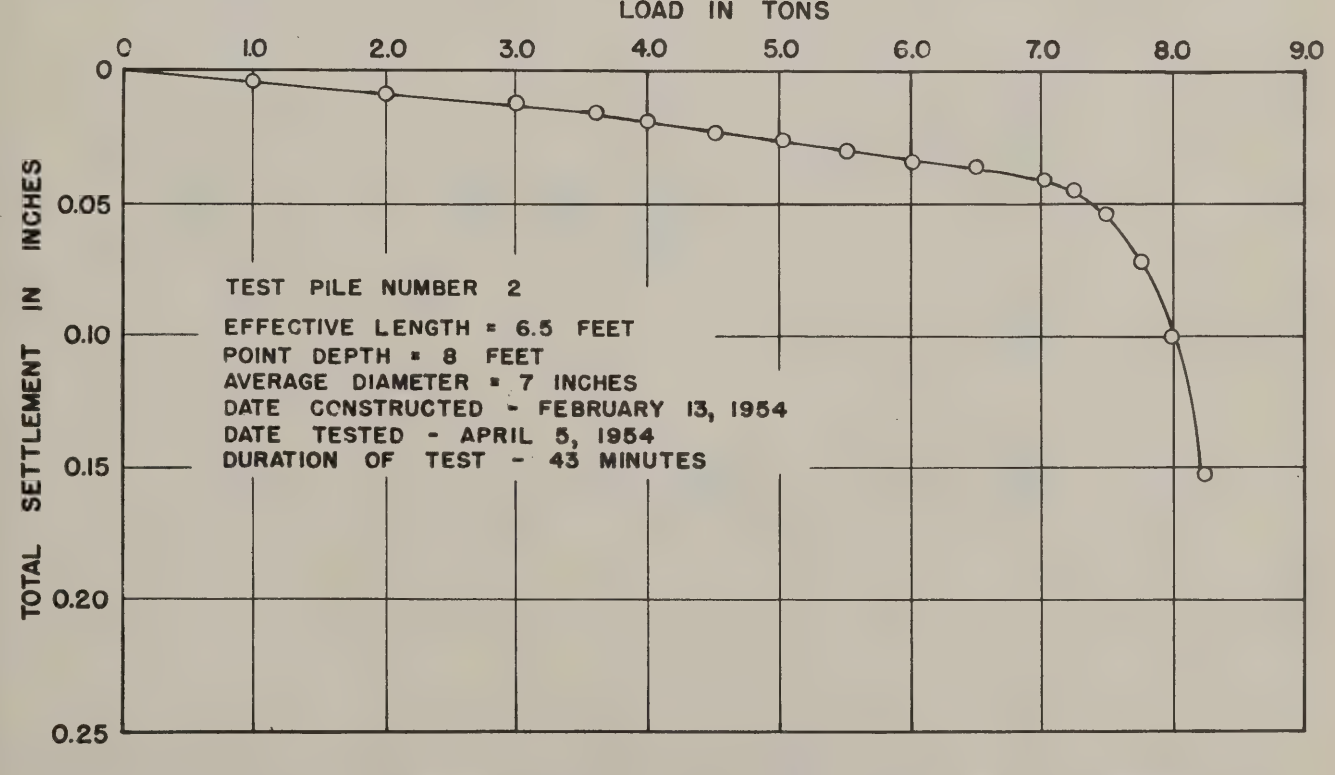


FIGURE 7 - LOAD-SETTLEMENT CURVE FOR "QUICK" TEST ON PILE NUMBER 2.

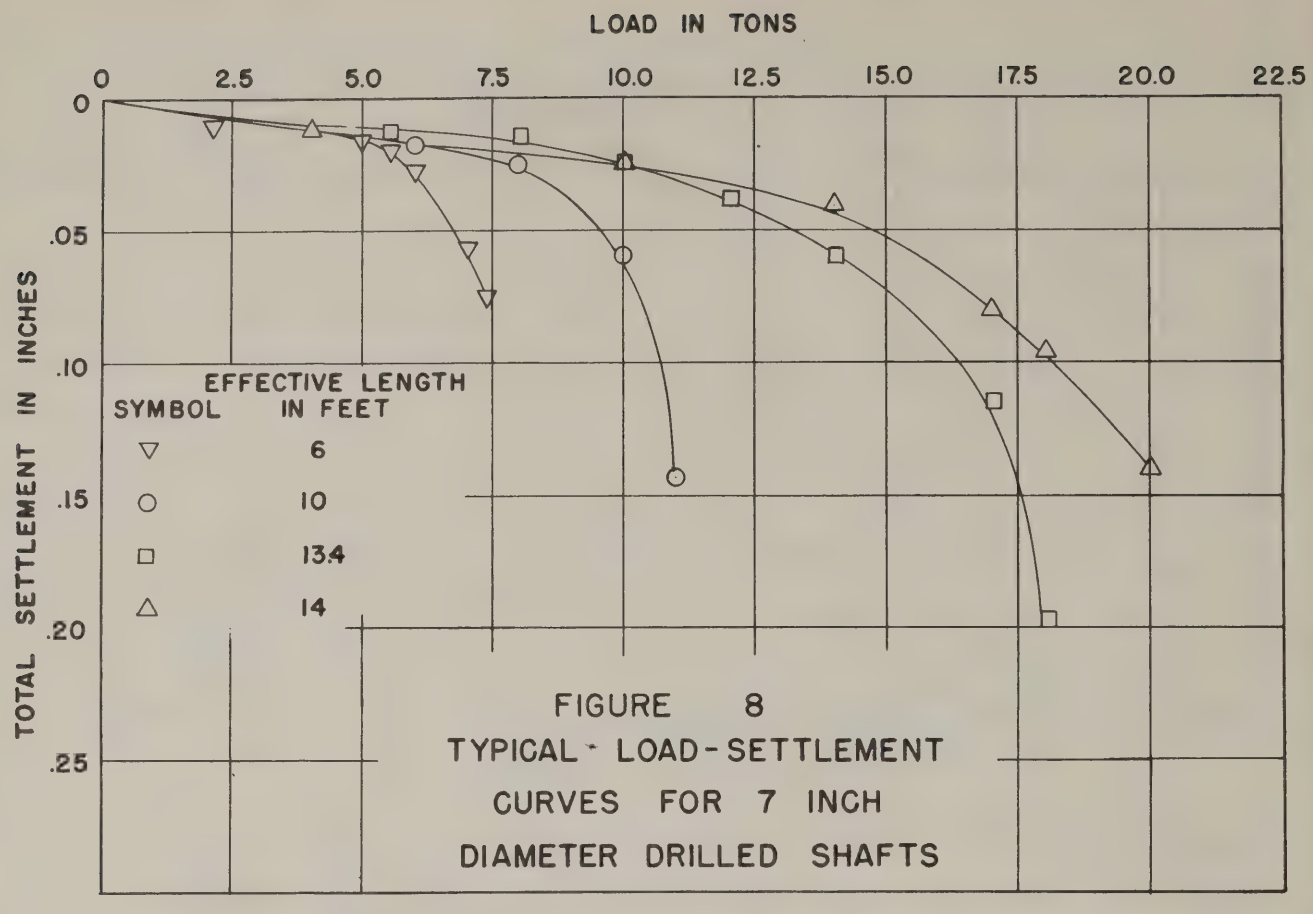


FIGURE 8
TYPICAL LOAD-SETTLEMENT CURVES FOR 7 INCH DIAMETER DRILLED SHAFTS

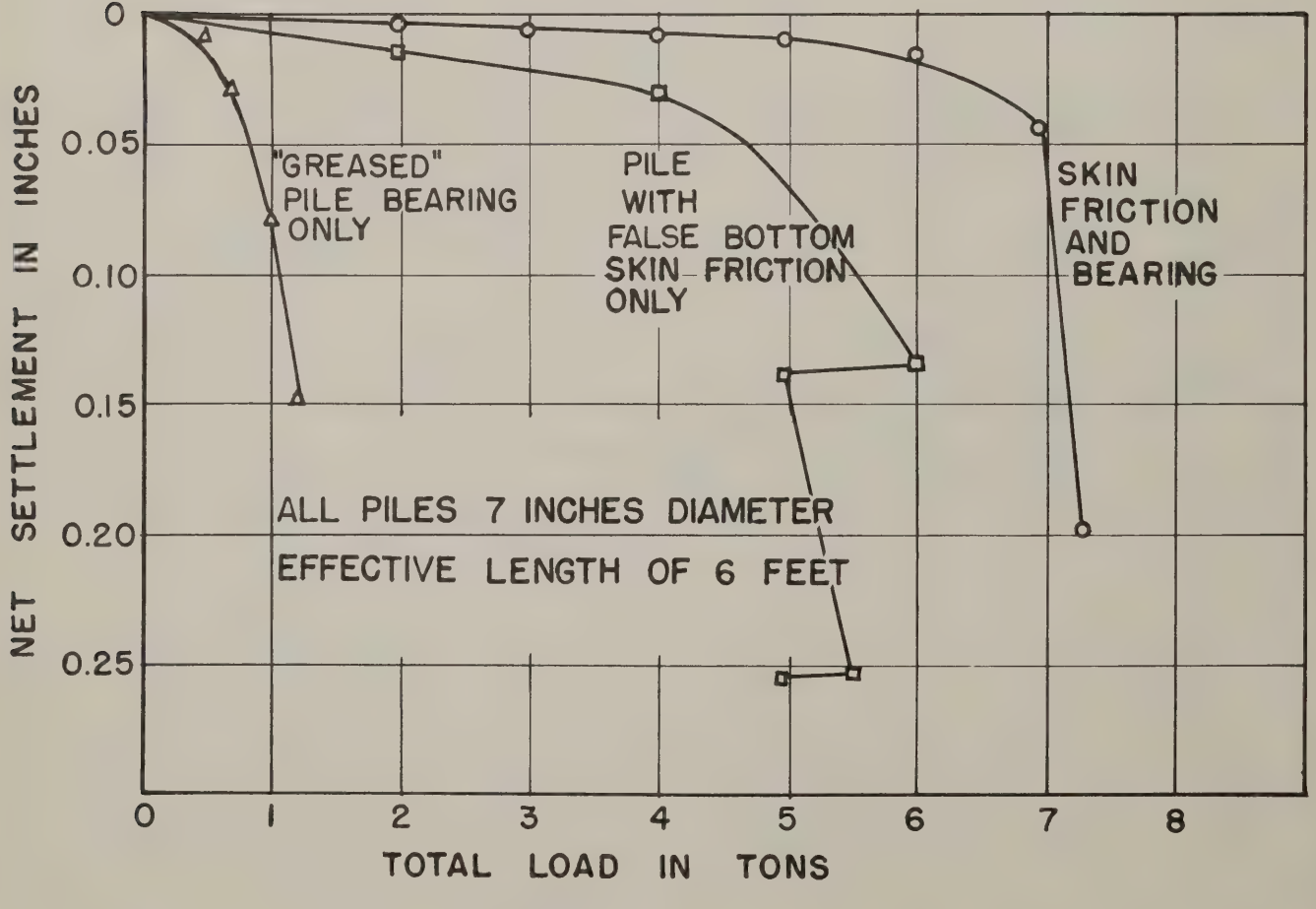


FIGURE 9

on a ten-foot effective length pile is shown in Figure 6. For this pile, the ultimate capacity by the Texas Highway Department load-test procedure, was considered to be ten tons.

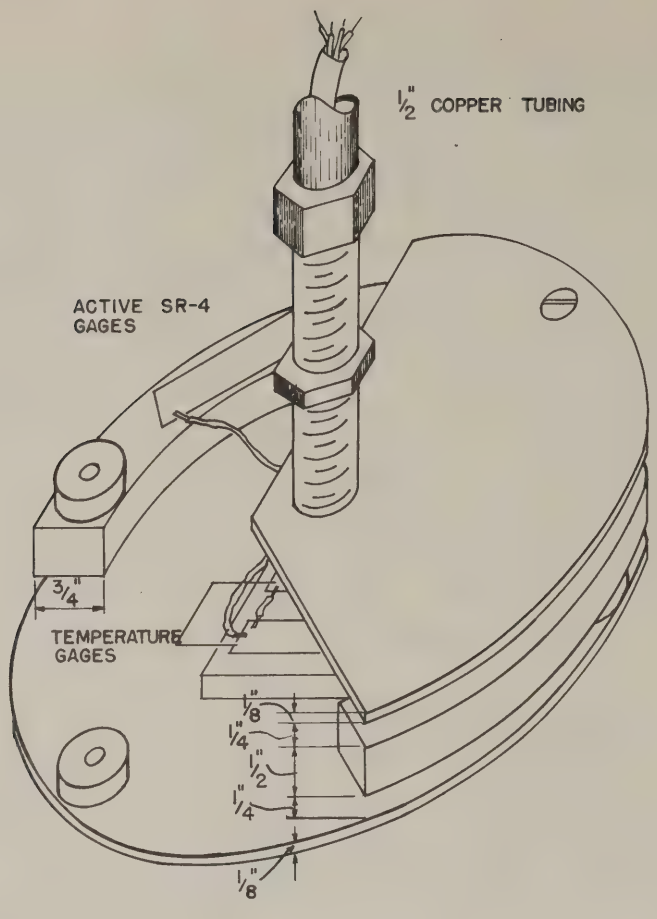
After the regular test was completed, the pile was permitted to recover, usually for a 24-hour period, and quick test performed. Loads were applied in small increments and the test continued until the pile continued to settle under a given load. These tests usually failed at a load about 10 to 15 per cent greater than the load given by the regular test method. An example of such a test is shown in Figure 7.

The data for the seven-inch diameter tests was considered as very good, despite premature failures for one of the fourteen-foot piles and for the eighteen-foot pile. Load-settlement curves show a significant increase in load-carrying capacity as the effective length increases. (See Figure 8.) Approximate ultimate load values of six, ten, eighteen, and twenty tons were obtained for piles with effective lengths of 6, 10, 13.4, and 14 feet, respectively. It is important to note that these were seven-inch diameter piles and the bearing area was the same for each test. Some of the increase in pile capacity can be attributed to additional base resistance resulting from the greater soil strength with respect to depth, however, most of the pile capacity increase is considered to be a result of the additional skin friction.

Also included with the tests on seven-inch diameter piles were three piles constructed under different conditions. The first test was on a six-foot effective length pile with the load being carried by a combination of skin friction and bearing. The load capacity for this pile was about seven tons. The second test has been referred to as the greased pile. A seven-inch diameter hole was augered to a depth of eight feet. Then a ten-foot section of six-inch diameter Sonotube which had been thoroughly greased inside and outside was placed in the hole and filled with concrete. The greased Sonotube eliminated almost if not all of the skin friction, thus giving a pile depending on bearing alone for load supporting ability. This pile was test loaded at about one ton. Completing this special study was a pile with a false bottom. This was constructed by bolting a plywood form to a piece of galvanized pipe. The pipe was secured at the surface so that a clearance of about two inches existed between the bottom of the boring and the form. This pile could only support load by skin friction. The test-load capacity was between five and six tons. The results of this study are shown in Figure 9.

Pressure Devices: One of the other important problems in this study was to develop a device for measuring bearing pressures during a test. Attempts were made in the laboratory to perfect devices based on both hydrostatic pressure and electrical resistance. Only the latter was developed to a degree to permit satisfactory field use.

This pressure device consisted of two plates separated by a center ring with three one-quarter-inch-thick washers between each plate and the ring. By spacing the washers 120° apart and offsetting them 60° on opposite sides of the ring, three tension areas could be created on each side of the ring. Three SR-4 gauges were cemented to the tension areas on one side of the ring and connected in series. Three additional gauges to serve as temperature gauges were cemented to an unstressed piece of metal in the center of the bottom plate. These were also connected in series and the four lead wires were brought out of the cell through a length of copper tubing attached to the center of



10 CELL FOR MEASURING PRESSURE AT BASE OF PILE

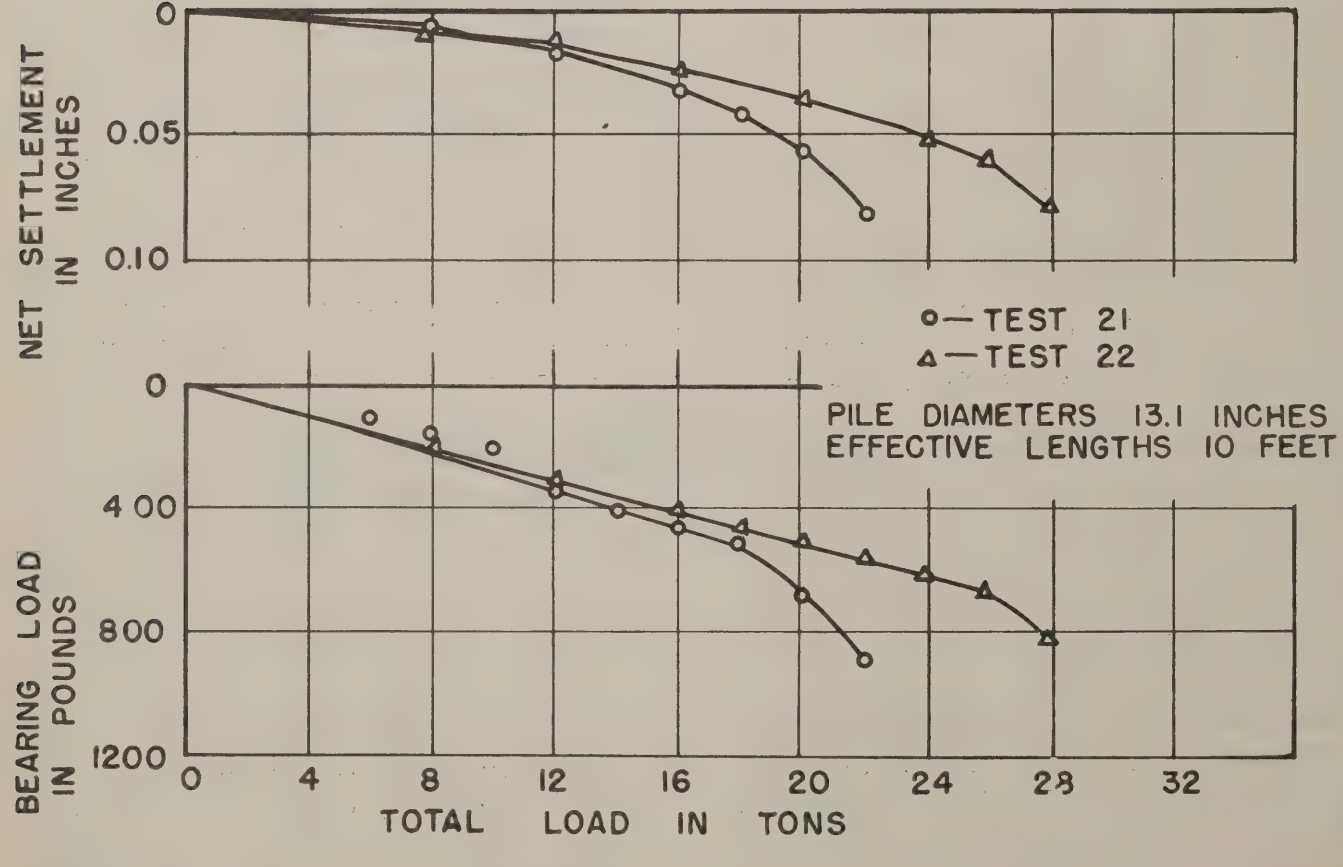


FIGURE 11

A COMPARISON OF SETTLEMENT AND BEARING LOAD TO TOTAL PILE LOAD

the top plate. The SR-4 gauges were waterproofed with an Ozite compound and then the cells were filled with a nonhardening Permatex. A relationship between load and resistance change was established by laboratory calibration.

Six of these cells (four of six-inch diameter and two of twelve-inch diameter) were made and installed for field tests. In each instance the cells gave consistent readings during the load tests. One test pile was extracted after completion of the load test and the pressure cell was removed from the pile and recalibrated. Essentially the same calibration curve was obtained.

Data from two of the 13.1-inch diameter tests are included to illustrate the consistency of these data. (See Figure 11.) Both piles had effective lengths of ten feet. Almost duplicate curves can be obtained for total load versus settlement and total load versus bearing load for both tests number 21 and 22. It is not unreasonable to assume that a large increase in bearing load would have resulted for an additional two tons of load in each test.

This paper is not the result of a completed study, but represents a progress report. Other studies concerning the influence of time on pile capacity, the comparison of straight shaft and underreamed piles, and moisture migration from the concrete into the soil and its effect on soil strength are planned for next year.

On the basis of work completed, the tentative conclusion can be made that skin friction has played a major role in supporting load for the piles tested.

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6. Plan Preparation Book III, Bridge Division, Texas Highway Department.

7. Terzaghi and Peck, Soil Mechanics in Engineering Practice, J. Wiley, New York, 1948.

MAINTENANCE OPERATIONS

Social Room

Chairman: L. K. Jonas, Chief Supervisor Training Program
Assisted by A. L. Kramer, Field Instructor
Texas Engineering Extension Service
A & M College of Texas

9:00 Causes of Job Accidents

1:30 Prevention of Job Accidents

Note: This program is intended for Maintenance Foremen and Maintenance Engineers particularly interested in reducing lost time and personal injury accidents. Maximum number which can be accommodated is 30.

GEOMETRIC DESIGN AND TRAFFIC CONTROL

Morning: Room 2-D

Chairman: M. V. Greer, Supervising Traffic Engineer
Texas Highway Department
Austin, Texas

9:00 Traffic Accident Data in Highway Design and Operation
W. R. Welty, Traffic Designing Engineer
Texas Highway Department
Austin, Texas

10:30 Use of Traffic Signs
P. R. Tutt, Traffic Designing Engineer
Texas Highway Department
Austin, Texas

Afternoon: Room 3-A

Chairman: W. J. Van London, Engineer-Manager
Texas Highway Department
Houston, Texas

1:30 Expressway Design Standards
Douglas A. Nettleton, Supervising Urban Engineer
Texas Highway Department
Dallas, Texas

3:00 Design for Expressway Drainage
Carl F. Izzard, Chief Hydraulics Research
Bureau of Public Roads
Washington, D. C.

TRAFFIC ACCIDENT DATA IN HIGHWAY DESIGN AND OPERATION

W. R. Welty, Traffic Designing Engineer
Texas Highway Department
Austin, Texas

As you well know, the traffic accident problem is of major concern. Practically every radio broadcast, almost every speech before a luncheon club, every newspaper, and every other form of public information all proclaim the usual fact that yearly more persons are dying in traffic accidents. Without in any way wishing to minimize the seriousness of the accident problem, it should be pointed out that since 1951 the number of persons who have been killed each year in traffic accidents has decreased a few hundred.

It is believed that the "scare" technique of presenting accident data will not accomplish its desired purpose of reduction in accidents because the motorists appear not to be impressed with stories and pictures of gory wrecks.

Since the number of persons killed each year in traffic accidents has, in general, shown an upward trend except the period since 1951, it might at first glance appear that all the traffic safety activities have come to naught. Actually, it is only to be expected that as more mileage is driven, there is a greater likelihood that traffic accidents and, therefore, fatalities will increase. In this connection, it is heartening to note that the mileage death rate per 100 million vehicle miles driven has dropped steadily from 16.6 in 1935 to 6.4 in 1953. It may be inferred from these facts that the three E's of traffic safety (i.e., Engineering, Enforcement, and Education) have played a major role in this reduction.

Traffic accident records are the most important source of the data which is used to interpret accident causation in its relation to various traffic safety activities. These records are kept by official agencies such as city police departments, sheriffs' departments, and state agencies such as the Texas Department of Public Safety. The reports are confidential and information thereon may be used only for traffic safety purposes by officials directly responsible for furthering such programs. In no case may the accident data be admitted as evidence in either criminal or civil litigation.

The Uniform Act Regulating Traffic on Highways, which has been adopted by the State of Texas, requires every driver of a motor vehicle involved in an accident which results in injury or death of any person or total property damage to an apparent extent of $25 or more to forward a written report of

such accident to the Department of Public Safety in Austin, Texas.

The Department of Public Safety processes the accident reports and then proceeds to analyze the data by means of punch cards so as to develop state-wide nonlocation statistics which are of interest to persons engaged in enforcement, education, and, to a lesser extent, engineering phases of traffic safety.

The engineer is primarily interested in traffic accidents from the standpoint of accident prevention through design of facilities. Accident data can furnish information which can be used to improve the accident record of existing highways through redesign; more effective use of signs, signals, markings, and islands; or use of accidents in planning and showing the need for improvement of the more deficient sections of highway. By pointing up the deficiencies in an existing facility, public support for an action program to secure right of way and additional sources of revenue for constructional and maintenance purposes can often be secured.

The biggest stumbling block in the use of accident data on a state-wide basis has been a lack of a means for establishing a correlation between the accident and the design features. To provide this correlation, accurate location data are required. Unfortunately, many of our accidents occur where definitive landmarks are lacking. For this reason, if for no other, the milepost mileage should be placed on the route marker posts as outlined in the Texas Manual on Traffic Control Devices.

The highway patrolmen are naturally more careful to reference the accident with respect to landmarks which are easily identified on maps and road logs than are motorists who, more than likely, have no idea that pinpointing the accident location is of such importance. Such features as the intersection of state designated and marked routes, county lines, railroad grade crossings or separations, and bridges over named watercourses which are shown on highway maps and road logs are desirable features from which an accident can be referenced. City limit lines are a very poor substitute for landmarks of a character such as are listed above in view of the fact that city limit lines are subject to change at any time and we are unable to keep our maps and road logs current in this respect.

Graphic methods of presentation of accident data are quite useful to the engineer. A spot map shows at a glance locations or sections of highway having high accident frequency. The "spots" may by color or design classify the accidents by their severity, and/or they may be further subdivided as to whether the accidents occurred at night or during the day, whether the accidents involved motor vehicles alone or pedestrians. In general, however, we believe that too much detail for spot map information may defeat the purpose of the spot map. The spot map, in our opinion, should be used as an index or an over-all view of the accidents in a given area and call attention to areas needing further special study.

After the spot map has been made, the accidents from those locations or sections of highway exhibiting apparent accident proneness should be studied and mileage rates computed. The mileage rate computation will allow comparison of the accident rate at that location with county, state, national, or other areal subdivision to see if the accident proneness is apparent or real.

More detailed accident studies usually involve the preparation of a collision diagram which shows diagrammatically the various factors involved in the various accidents depicted. In addition to the collision diagram, a more or less detailed plan of the accident area which shows all traffic-control devices, traffic regulations, and natural and man-made features such as sight distances, access points, grades, turning radii, pavement surfaces, etc. should be prepared. This plan is called a condition diagram. After the condition and collision diagrams have been prepared, the engineer should take them with him to the scene of the accidents. If, for example, right-angle accidents appear to predominate at an intersection from two particular directions, reference to the condition diagram and at-the-scene study of the problem may reveal that there is a sight restriction between the two directions involved. Accidents which occur about the

same time of day during periods of daylight might be found in the on-the-site study to be influenced by confusing shadows which occur during that time. In our experience, we have found a location where an oil derrick made a shadow at certain times of the day that apparently caused some motorists to run into the end of a bridge. Quite often inexplicable night accidents at traffic signal controlled intersections become understandable when it is found during a night observation that a red or green neon advertising sign is interfering with the signal indications.

Where head-on and sideswipe opposite direction accidents predominate on a two-lane, two-direction roadway, studies should be made to determine whether or not a divided highway could be justified at the location.

Accident data may be extracted by the "strong-arm" method (i.e., looking through the files) when the accident records are filed by, say counties. The strong-arm method is so time consuming, tedious, and expensive that its application must be limited to a very few locations per study, thereby making the information yielded too sketchy for any comparative analyses. For these and other reasons, mechanical sorting of coded cards seems to be the only practical method at the present time that will furnish information on a state-wide or other large areal basis. In cities, the accident records may be filed by location so that if information is needed at a specific location, only those accidents which have occurred at that location need be studied. Location files are impractical on a state-wide basis.

The Highway Department has recently instituted a system for coding data obtained from accident records on punch cards. These data are coded to locate the accidents on the highway by their distance, to the nearest one-tenth mile, measured from the north or west county line or from an intersection where the road begins. The data are also coded to give all necessary information about the accidents that might be used for engineering purposes, including the preparation of collision diagrams. Mechanical sorting will separate the cards by control and section and will arrange the data so that the accidents are placed in their proper mileage sequence down a particular highway. After the cards have been placed in the proper mileage sequence, they may be mechanically tabulated. From this mechanical tabulation, those highway locations having design or maintenance deficiencies become readily apparent.

Deficiencies in the roadway are contributory to accidents; although, the extent to which the roadway's deficiencies contribute to accident causation is a moot question. Some authorities place road defects as low as 2 per cent and some as high as 20 per cent. All of these figures are too low in our estimation because they only consider a road defect to be something of the nature of a hole in the roadway, but it appears logical that deficiencies in design are determining factors in highway accidents since the accident rate on a road having a high type of design like the Gulf Freeway (i.e., within the city limits of Houston) has a fatality rate of 1.6 fatalities per 100 million vehicle miles whereas the average death rate on Texas highways in 1953 ran 6.4. Incidentally, this 6.4 is the lowest rate recorded state-wide in Texas' history.

Accident records have been used in California to show that narrow medians are more dangerous on heavily traveled highways having more or less unlimited access from roadside development than are undivided highways in the same category. In Connecticut the Highway Department uses accident data for scheduling maintenance programs. For example, sections of roadway which have an unusually large number of skidding accidents receive a new high coefficient of friction surface treatment. A section of roadway may be found to have more than the average number of accidents in which the type and state of maintenance of the shoulder is a factor. Obviously, shoulder design and maintenance betterments are indicated.

This subject is so vast and its possibilities are so numerous with many of them either unknown or at least unappreciated that you can readily appreciate that I have only most sketchily outlined the subject.

THE USE OF TRAFFIC SIGNS

P.R. Tutt, Traffic Designing Engineer
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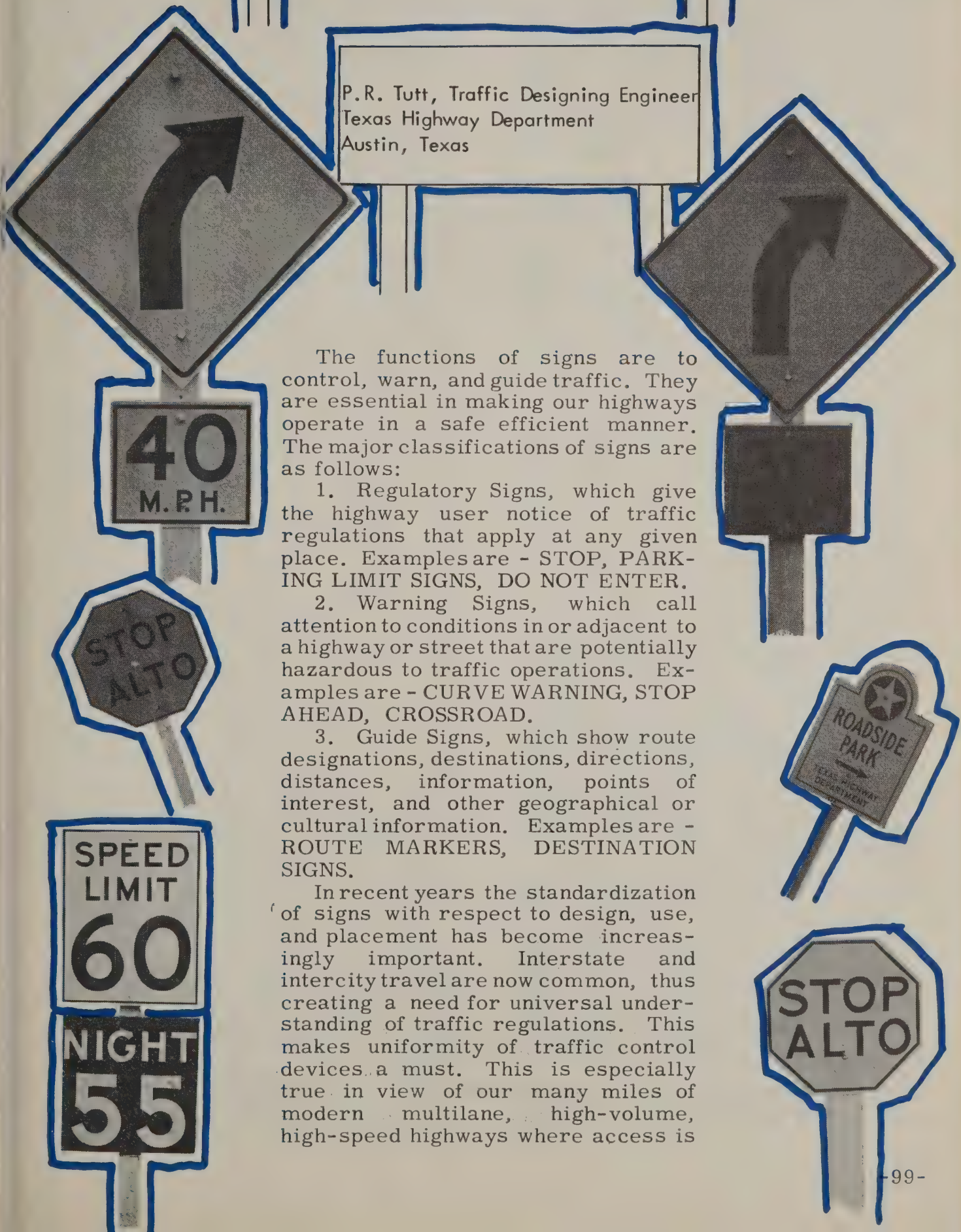

The functions of signs are to control, warn, and guide traffic. They are essential in making our highways operate in a safe efficient manner. The major classifications of signs are as follows:

1. Regulatory Signs, which give the highway user notice of traffic regulations that apply at any given place. Examples are - STOP, PARKING LIMIT SIGNS, DO NOT ENTER.

2. Warning Signs, which call attention to conditions in or adjacent to a highway or street that are potentially hazardous to traffic operations. Examples are - CURVE WARNING, STOP AHEAD, CROSSROAD.

3. Guide Signs, which show route designations, destinations, directions, distances, information, points of interest, and other geographical or cultural information. Examples are - ROUTE MARKERS, DESTINATION SIGNS.

In recent years the standardization of signs with respect to design, use, and placement has become increasingly important. Interstate and intercity travel are now common, thus creating a need for universal understanding of traffic regulations. This makes uniformity of traffic control devices a must. This is especially true in view of our many miles of modern multilane, high-volume, high-speed highways where access is

partially or fully controlled. A driver on one of these facilities usually has only a few seconds to observe a sign and must interpret its meaning immediately if he is to proceed safely and arrive at his destination without traveling in a circuitous route. A wrong turn or missing a turn on a controlled access facility might take a driver several miles out of his way.

Thus if different signs are used for a particular purpose in different cities or states, drivers will become confused and the result will be congestion and accidents.

The sign is the most common and most widely used of the traffic-control devices. It is commonly thought of as inexpensive, but due to the large number used, the bill runs rather high. In 1953 about 116,000 signs were erected by the Department. About $1,799,000 is spent every year on signing and sign maintenance. These figures include only the activities of the State. Many additional dollars are spent by cities. A simple crossing of two U.S. or State-numbered highways with only one highway on each road requires a minimum of 28 signs. Any sort of a special condition would probably increase this number and a channelized intersection will add many signs, depending on the type of channelization.

On the other hand, a single sign is such an inexpensive thing in comparison with other items of highway construction that in some cases we become too free in their use. This is expensive as well as detrimental from the standpoint of effective traffic control since too many signs can be as confusing to the traveling public as too few signs. The overuse of regulatory and warning signs is particularly bad because it causes drivers to be unimpressed with their importance and to often disregard not only the unneeded signs, but the needed ones as well. The overuse of guide signs is not objectionable from this standpoint, but it is wasteful and a few well-chosen signs will accomplish the desired result with less confusion to the public.

The new Sign Manual, which I am sure all of you are more or less familiar with, was designed to bring Texas as near as possible into conformance with the other 47 states in traffic control. It was patterned after the National Manual, and in most cases conforms to the national practices (which were developed through cooperation of all the states) and should represent the most efficient signing practices possible. There were a few instances where our previous practices were retained for one reason or another, but in all important respects the national practices were followed.

The manual shows typical signing plans for the more common types of interchanges such as crossings and Y and T type connections. Several examples of signing for more complicated interchanges are also shown, but modern highway design has produced such a variety of intersection types that it would be impossible to show examples which cover all situations that might come up. The general pattern is set, but there will be many cases where considerable ingenuity on the part of the man working out a specific signing plan will be required.

In signing unusually complicated interchanges such as the one of U.S. 81 with U.S. 190 and S.H. 317 in the south edge of Belton, the most important thing to remember is that the signing comes in steps as does the channelization. You can take the route of a particular highway through the interchange and draw it as a straight line. The various turns will then come off just as turns off any straight highway and will be signed in the same manner as turns from a straight road. For instance, taking the route of U.S. 190 eastbound through this interchange, we first come to the point where U.S. 81 to the south is turned off to the right. The next turn is S.H. 317 to the left. This is also signed as U.S. 190 Business Route. The next turn is for U.S. 190 westbound which turns off to the left. With the road drawn straight it is a simple matter to locate the required signs. This process repeated for each highway through the interchange will complete the basic signing plan.

In an intersection such as this,

where the traffic movements in opposite directions on a route are not always parallel to each other, Cardinal Directional Markers are very important. These are the small signs used in conjunction with route markers which indicate the general direction of the particular route, such as northbound or westbound.

Advance route markers are used here as they would be at any intersection, but due to the proximity of the turns and the various aspects of the interchange which tend to distract the drivers, their effectiveness may be somewhat reduced. It may also not always be possible to place the advance marker as far in advance of the actual turn as is required by the speed of traffic because it might be confused with signs for some other turn. All of this tends to cause the drivers to place more reliance on the signs at the turn. For this reason oversize destination signs were used at this particular location. I believe the letters in this case were increased from seven inches to twelve inches in height. The exact position of the signs should also be studied carefully so that they will be visible for the greatest distance in advance of the turn.

Another possibility in signing complicated interchanges is the use of an advance informational sign which shows an approximate diagram of the interchange as it applies to the driver on a particular approach. Here is an example of how this can be applied to a traffic circle. A driver approaching this circle and wishing to leave on U. S. 87 sees this sign. From it he can tell that he must travel a little more than half of the way around the circle before coming to his exit point. Knowing this he will drive with more confidence and will be able to get into the outer lane at the proper time to make his turn.

The use of this type of sign is optional, and due to its tremendous size should be limited to trouble spots. The ever-increasing use of complicated interchanges is bound to produce a need for more of this type of signing, however.

The signing of expressways and freeways often presents problems not encountered in ordinary highway signing. The extremely heavy traffic volumes encountered on expressways often make it difficult and distance consuming to change lanes. For this reason expressway drivers must be notified of exit points a considerable distance in advance of those points so that they can get into the proper lane to get off the expressway if they so desire. The placing of these advance notice signs is especially important because they must be seen by traffic in all of the two, three, or four lanes which make up one direction of flow on an expressway. This may indicate the need for an overhead sign or at least a sign placed high enough that it can be seen over the tops of other vehicles. Signs of this type may also require illumination because they are far enough out of the direct path of traffic that reflective sheeting may not be effective. Overhead signs, mounted on supports on either side of the roadway and suspended directly over the roadway are particularly effective for this sort of thing because they are located directly in front of the driver. He can hardly avoid seeing them no matter how busy he is watching traffic.

District 12 has several of these overhead signs at important interchanges on the Gulf Freeway.

There is also another, if somewhat indirect role, which signs can play in the building of a more efficient highway system. The working out of the signing plan in conjunction with the design of an intersection can be very helpful in achieving a more efficient design.

The usual procedure seems to be to complete the construction of an intersection before signing is considered. A layout is then turned over to the man in charge of signing and he is instructed to properly place the signs. In working out the signing it is almost probable that several points will come up where slight or possibly major changes in the design would improve not only the signing, but the general operation of the intersection. The location of curbs and curb returns can often be varied with little or no effect on the design, and their exact location may make considerable difference in the effectiveness of the signs.

Working out the paths which the signs will direct traffic to follow through the interchange may also bring out points which had not been considered before and which might improve the design considerably. The only trouble with this is that the intersection is usually completed by the time this is discovered and the engineer in charge will undoubtedly be reluctant to make changes - changes which could have been made with no difficulty had they been suggested before the intersection was constructed. The point I'm trying to make here is that if the signing experts and the intersection designers could work more closely on these intersections, we would probably wind up with not only better signing but better designs also.

This is also true of pavement markings, which may be a little off the subject, but it is important enough that I believe it is worth mentioning. The most frequent difficulty which placing pavement markings on a design is apt to show up is an offset in the lanes through a transition from a curbed section to a noncurbed section. The pavement in the curbed section is usually slightly wider than that in the section which is not curbed. This is to make allowance for the fact that drivers will not drive as close to a curb as they will to a shoulder.

In laying out the intersection, the designer often merely lines up the pavement edge on one side or the other. When the lanes are marked through such a transition, we find that the lane lines on either side of the transition do not line up, making it necessary to use a reverse curve or sometimes a section of tangent to connect the two lane lines. This is certainly undesirable, to say the least. If the lane lines had been placed on the drawing before the intersection was constructed, this condition would have been discovered and remedied at no cost and very little inconvenience.

It may be that some of the districts already work out both signing and marking plans for intersections before they are constructed, but some apparently don't because I have seen several plans recently where the offset which I mentioned existed. A condition such as this is virtually impossible to correct once it has occurred because so much pavement is involved and it is difficult to add a narrow strip in such a manner that it will be used by traffic.

Another important use of signs is as an accident-prevention measure. In one sense all regulatory and warning signs are intended to prevent accidents, but many are erected as standard signing procedure and not specifically for the prevention of accidents. An accident-prone location is usually caused by a design deficiency rather than a signing deficiency, but it is often more expedient and usually cheaper to correct the situation with signs, if this is possible. Signs may be a poor substitute for design changes, but they can be erected quickly and are often very effective in preventing accidents. These signs may take several forms. One possibility would be an unusually sharp or deceiving curve where a large turn arrow might be used to supplement the standard markings. Another might be an auxiliary or oversize STOP sign where drivers have not been observing the standard signs.

Signs as accident prevention measures will often be temporary, to be used until design changes can be made, but they are very important in this role. Their use should be based on a study of accident records and of the physical conditions at a trouble spot. If these studies indicate that additional signs or changes in the existing signs might prevent the accidents, the most likely procedure to follow would be to try this first. If the signs are not effective, more drastic steps such as changes in the design should be taken.

The importance of the role which signs play in the operation of our highways cannot be overemphasized, and the necessity for having well-designed, clearly understandable signs is increasingly evident. The key to this is uniformity; uniformity in the design of the signs; uniformity in the location of signs, and uniformity in the application and use of signs.

DESIGN STANDARDS FOR EXPRESSWAYS

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TRAFFIC ANALYSIS: Before any highway can be intelligently designed, the amount and distribution of traffic it is expected to carry should be determined. In the case of an expressway in a large urban area, this determination is a complex problem. The first step is an Origin and Destination survey, but that is just the beginning. Less susceptible to exact analysis is the assignment of traffic to the expressways, and the prediction of the future increase in traffic. A number of methods of assignment have been used, but probably the simplest and best is on the basis of the ratio of time via the expressway to time via the best alternate route. Studies (1) made on a number of operating expressways have resulted in an S-shaped curve which shows that 90 per cent of total trips will use the expressway for a time ratio of 0.6, 48 per cent for a time ratio of 1, and 10 per cent for a time ratio of 1.33. In order to get the time for each trip, distances may be scaled from a map and speeds assumed, or actual time runs may be made for the existing routes. A metropolitan area is generally divided into several hundred zones in the O. & D. survey and the resulting number of combinations of trips between zones may be in the hundreds of thousands. Each of these combinations must be analyzed for possible assignment of traffic to an expressway.

In Dallas, an O. & D. survey was made by the Planning Survey of the Highway Department, and the City Traffic Department made an allocation of this traffic. The City Traffic Department projected this allocation to 1980 by analyzing the rate of population growth, increasing car ownership and increasing car usage. They came up with a figure of 2.05 for the average ratio of traffic in Dallas in 1980 to that in 1950. It is interesting to note the relationship between traffic originating in or destined for the central business district as compared to the total traffic. On the average, only 27.5 per cent of the expressway traffic has its origin or destination in the central business district. The remaining traffic, which is 72.5 per cent of the total, must transfer from one radiating expressway to another in order to reach its destination. In large cities, where there are a number of radiating expressways, an expressway loop around the central business district is a necessity to effect this transfer without using overcrowded city streets.

For design purposes, the 24-hour traffic, as determined by allocation of the O. & D. study, must be reduced to peak-hour flow in one direction. The thirtieth highest hour of the year is generally used for design purposes. Urban highways have lower peak-hour percentages than rural highways. The national average percentage of annual average daily traffic for the thirtieth highest hour of the year is 16.2 on rural multilane roads and 11.6 on urban roads. (2) These percentages are for the total traffic in the two directions and must be broken down into one-direction traffic for multilane roads. This breakdown is, on the average, about 2/3 and 1/3. Thus urban expressways, on the average, should be designed for an hourly traffic in one direction equal to 2/3 of 12 per cent, or 8 per cent of the two-way 24-hour traffic. A study of the actual traffic counts on Central Expressway shows this percentage to be 8.2.

CAPACITIES: Lane capacities on urban freeways are generally taken, for design purposes, as 1,500 passenger vehicles per hour on 12-foot lanes. (3) This volume of traffic tends to move at an average speed of only about 36 miles per hour. (4) If for any reason the traffic speed is reduced, the same volume can be carried at a speed as low as seventeen

miles per hour. It is possible to carry as many as 2,000 passenger vehicles per hour per lane, but the speed is then at a critical point of about 32 miles per hour. Traffic can move very little faster or slower than this speed and still remain at such a large volume. If the traffic is slowed for any reason, the highway capacity is less than the demand and a traffic jam results. It is not, therefore, advisable to design for this high possible volume. The presence of trucks will reduce the design capacity as much as 25 per cent for a ratio of 20 per cent trucks and rolling terrain. (5) Where merging of traffic is taking place, the design capacity of a lane is reduced to 1,200 passenger vehicles per hour if it is desired to keep the speed at about 35 miles per hour. At lower speeds, it is possible to merge 1,500 passenger vehicles per hour. (6)

A recent traffic count on Central Expressway during the morning rush hour from 7:30 to 8:30 showed 5,318 vehicles in three lanes, which is an average of 1,773 vehicles per lane. Over a twenty-minute period, the average per lane was at the rate of 1,936 vehicles per hour. The men making the count estimated that the traffic was moving at an average speed of 40 miles per hour. This would indicate that the cars were spaced closer than is generally considered safe for this speed. It is interesting to note that the maximum traffic on Central Expressway occurs about two miles out from the edge of the central business district and is about 10 per cent higher than that at the permanent traffic counter which was placed closer in, at a point where it was thought that traffic would be a maximum.

DESIGN SPEED: Since a considerable percentage of the usage of an urban expressway is under high-volume conditions of 1,000 to 1,500 vehicles per lane per hour, for which the average running speed is between 35 and 40 miles per hour, a design speed of 50 miles per hour is ample. Even during periods of low volume, when higher speeds are possible, the average length of trip is so short that the saving in time for a 60-mile-per-hour speed as compared to a 50-mile-per-hour speed is negligible — just one minute for a five-mile trip. It is not necessary to base all phases of the design of an expressway upon the full design speed. Features of the design which are critical only under heavy volumes of traffic should be based upon the average running speed under this heavy volume. Length required for weaving and merging of traffic fall in this category. The new A.A.S.H.O. Policy Books use a 40-mile-per-hour average running speed when the design speed is 50 m.p.h.

WEAVING LENGTH: Weaving movements should be avoided wherever practicable. The term weaving, as used here, refers to the crossing of two streams of traffic in a relatively short distance by first merging and then diverging. Weaving occurs most frequently on urban expressways in connection with interchange movements. For example, at a cloverleaf interchange the on- and off-bound traffic must cross in the distance between the inner loops. Similarly, diamond-type interchanges which are close together should be investigated for weaving movements between the on- and off-bound traffic of consecutive interchanges. The "Highway Capacity Manual" has a chart giving the required length of weaving section for various numbers of weaving vehicles at different operating speeds. The number of weaving vehicles, in this chart, is the sum of the two conflicting flows. If the sum of on- and off-bound movements in the same direction at the same time is 1,000 vehicles per hour, the weaving distance required by this chart is 500 feet for a 40-mile-per-hour operating speed. Weaving sections are considered practical only where the total number of vehicles required to weave does not exceed 1,500 vehicles per hour. Thus, two heavily loaded expressway lanes should not be required to weave with each other. Weaving can generally be eliminated by the use of one or more grade separation structures. Even a very small volume of traffic, entering the expressway from one side, should not be allowed to weave across the expressway in a short distance to an exit on the other side.

MERGING LENGTHS: Merging occurs wherever there is an entrance to the freeway. Lengths of acceleration lanes, as given in

the new A.A.S.H.O. Policy Tables, are based entirely upon the distance required for acceleration, and are simply a function of the design speeds of the approach roadway and the freeway. For a 50 m.p.h. design speed, these lengths vary from 250 to 700 feet, depending upon the approach speed. Where the entering vehicle moves from a parallel frontage street into the freeway, these two design speeds might be taken as 30 and 50 miles per hour respectively, and the table would show 500 feet as the required length. In calculating this distance, no allowance is made for the fact that the entering driver may very likely encounter a string of cars on the freeway traveling too close together to allow him to safely enter between them.

PROPOSED THEORY FOR LENGTHS OF ENTRANCE LANES: The following is a proposed theory for lengths of entrance lanes, which takes this condition into account. Observations made with motion picture cameras indicate that entering drivers require a gap of between one and three-fourths and three seconds in length to merge with vehicles moving at approximately the same speed. (7)

The spacing of gaps of any desired length can be determined by the laws of probability, but we must assume some acceptable degree of probability. The figures worked out in this paper are based upon 98 per cent of the entering drivers finding gap conditions at least as favorable as that upon which the design is based. The remaining 2 per cent of entering drivers would be forced to delay at a very low speed or stop completely.

It can be shown that on the basis of this 98 per cent probability and a volume of traffic on the outer lane of the freeway ahead of the entrance equal to 1,000 vehicles per hour, the entering driver may have to allow four unacceptable gaps to pass before an acceptable gap of two seconds appears. (8) It can also be shown, by the laws of probability, that the average length of these unacceptable gaps is 0.9 seconds. (9)

If the average running speed on the freeway is 40 miles per hour and if the entering driver accelerates, at a uniform rate, from 20 to 35 m.p.h., the two-second gap will overtake him in 12.65 seconds and he will have traveled 510 feet. (10) He may then begin his lateral movement onto the freeway lane. Observations have indicated that a distance of about 200 feet is required for this lateral movement when moving at this speed. If he accelerates at the normal rate of acceleration during this lateral movement, he will be moving at 39 m.p.h., and will still be a safe distance of 82 feet ahead of the car behind him at the end of the lateral movement. Thus the total length of acceleration lane is 510 plus 200, or 710 feet — in round numbers, 700 feet. A length of 500 feet would be sufficient to permit 86 per cent of the approaching vehicles to enter without stopping or slowing to a very low speed. The length of lane should be measured from the point where the entering driver can look back and see the approaching suitable gap, which would be 330 feet behind him for the 700-foot length just worked out.

Although a starting speed of 20 m.p.h. has been assumed, there is no reason why the entering driver could not be running at from 30 to 40 m.p.h. at the point where he can first look back on the freeway. In that case, however, he would have to first slow down and then speed up in order to allow the acceptable gap to overtake him. He could also start at a speed of 15 m.p.h., but in that case, would have to accelerate over part of the distance at a rate faster than normal.

Many of the entering lanes on urban freeways in Texas, including the early sections of Central Expressway, are too short, and have been the subject of some adverse comment. Many times on these short ramps the entering vehicles are forced to stop, and rear-end collisions frequently result because the driver behind may be looking back for a suitable gap in the freeway traffic. Or, accidents may be caused by impatient drivers trying to enter gaps which are too short, causing the freeway vehicle to brake suddenly or swerve into the adjacent lane. Of course, if the volume of entering traffic is very small, there is no particular objection to a short entrance lane which causes the majority of entering vehicles to stop and

wait for a gap that can be entered from a standing position. This length of gap has been observed to be about six seconds. However, it is better to eliminate those entrances which will carry only small volumes of traffic.

EXIT LANES: Exit lanes can be much shorter than entrance lanes. As far as the freeway is concerned, it is simply a matter of providing deceleration distance. Where the exit lane joins a one-way frontage street just ahead of an intersection, it may be desirable to add an extra lane on the frontage street up to the intersection, to provide storage and to avoid conflict with the through traffic on the frontage street. The distance given in the Policy Table for a deceleration lane between roadways having 50-mile and 30-mile-per-hour design speeds is 250 feet. This increases to 400 feet in going from a 50-mile-per-hour design speed to a required stop. These lengths include the necessary taper of 200 feet for the lateral movement out of the through lane.

CAPACITIES OF ENTRANCE AND EXITS: One-lane entrances should not be counted upon to take care of more than about 900 vehicles per hour. If it is desired to provide for appreciably more vehicles than this, a two-lane entrance should be used and a freeway lane added from that point on to where the traffic has been reduced sufficiently to drop a lane. A parallel line of reasoning applies to exits which must provide for more than 900 vehicles per hour.

JUNCTION OF ROADWAY: Sometimes two one-way roadways of two lanes each join and are tapered down to three lanes. If the roadways are of a similar nature and have equal design speeds, all of the traffic will be running at about the same speed. This problem is difficult to solve on the basis of finding acceptable gaps since the merging must be accomplished by complementary changing of speeds by the conflicting vehicles. The merging length for this condition should be at least the 700 feet worked out for an entrance lane.

HORIZONTAL ALIGNMENT: Standards for horizontal alignment, as given in the new A.A.S.H.O. Policy Books, show the following maximum rates of curvature for a design speed of 50 m.p.h.:

6.9° where the maximum superelevation is limited to 0.06 of a foot per foot
7.6° where the maximum superelevation is limited to 0.08 of a foot per foot
8.8° where the maximum superelevation is limited to 0.10 of a foot per foot

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These are based upon utilizing a side slip friction coefficient of 0.14. It is recommended that the maximum superelevation on urban structures be limited to 0.06 of a foot per foot because of rapid freezing and thawing due to direct exposure to air temperature beneath the deck and building shadows on the pavement. In fact, for reasons of appearance, the maximum superelevation on all urban roadways should be limited to 6 per cent. For a design speed of 50 m.p.h., superelevation is required for curves of more than 0 degrees 30 minutes. The amount of superelevation for curves intermediate between the maximum and the minimum is determined by a parabolic curve with its vertex at the point of maximum curvature and maximum superelevation. The minimum degree of curve requiring the use of a spiral, at 50 m.p.h., is 1 degree 45 minutes. The length of spiral is a function of degree of curve, maximum superelevation, design speed, and width of roadway. The new A.A.S.H.O. Policy Manual recommends that four-lane roadways have 1.5 times the length for two-lane roadways and that six-lane roadways have twice the length for two-lane roadways. This is to limit the rate of change of grade at the outer edges of the roadway. A 290-foot length of spiral is required for a six-lane expressway on a 6.9 degree curve with 6 per cent superelevation. Where the median is sufficiently wide, each roadway may be superelevated independently by rotating about its center line and the length of spiral is then reduced to 150 feet.

VERTICAL CONTROLS: Grades on urban freeways should be limited to a maximum of

4 per cent and the minimum grade for curbed sections should be 0.5 per cent for proper drainage. The relationship between length of grade and per cent of grade should be such that the speed reduction of heavy trucks will be not more than 15 m.p.h. below the average running speed. Where the design speed is 50 m.p.h. and the average running speed is 40 m.p.h., these limiting lengths are: 1,070 feet for a 4 per cent grade and 1,570 feet for a 3 per cent grade.

Lengths of crest vertical curves are based upon a sight distance to an object 4 inches high equal to the stopping distance. For a 50-mile-per-hour design speed, the stopping distance is 350 feet, including perception and reaction time, and the rounded value for length of vertical curve is equal to 80 times the algebraic difference in grades, measured in per cent with a minimum value of 150 feet. This gives 640 feet for length of vertical curve at the crest of two 4 per cent grades. This is the minimum, and longer vertical curves are desirable where feasible.

Lengths of sag vertical curves may be based upon a headlight sight distance equal to the stopping distance. Headlight sight distance is measured to the point where the direct beam of the headlight strikes the pavement. Assuming 2.5 feet for height of headlight and a 1 degree upward divergence, the rounded length of vertical curve for a 50 m.p.h. design speed is equal to 70 times the algebraic difference in grade with a minimum value of 150 feet. Where the highway is well lighted, the length of vertical curves in sags need not be based upon headlight sight distance, and the length should be based upon riding comfort. Assuming that centripetal acceleration should not exceed one foot per second per second, the length of curve should be equal to 54 times the algebraic difference in grade for a design speed of 50 m.p.h. Thus two 4 per cent grades would require a sag vertical curve length of 432 feet. This is the A.A.S.H.O. recommendation, but they state that more study should be given to this matter of riding comfort on sag vertical curves.

CROSS-SECTIONAL FEATURES: Expressway lane widths should be not less than twelve feet and the cross slope on through traffic lanes should be in the range of one-eighth to one-fourth of an inch per foot. The steeper slope is used in the lane adjacent to the water-carrying gutter. A minimum width of median of fifteen feet is desirable, but high cost of right of way may result in the use of median widths down to a minimum of four feet. A twelve-foot width will permit the use of a center pier at underpasses. It is a moot question as to whether median curbs should be mountable or nonmountable. The need for a positive barrier is greatest in the case of a narrow median, but the two-foot offset required for barrier curbs results in taking four feet off the median width. If mountable curbs are used and a barrier fence or wall is placed in the center of a narrow median, there may not be proper clearance between the barrier face and the face of curb.

Paved shoulders for emergency parking are a necessity on urban expressways. One stalled vehicle on the through pavement reduces the capacity much more than the value of one lane, and may very well result in a complete tie-up during peak hours. The shoulder should have a width of ten feet and should be separated from the traffic lane by a mountable curb four inches high, with its face making an angle of about 30 degrees with the horizontal.

FRONTAGE STREETS: Most frontage streets in Texas have been built to a width of 32 feet, allowing two twelve-foot travel lanes and an eight-foot parking lane. In general, frontage streets should be one way to avoid conflicts at the exits and entrances to the freeway. In many cases, a twenty-foot width may be sufficient for a one-way frontage street. This width lends itself to the addition of an extra lane to the frontage street at the interchanges, which, as previously mentioned, is desirable to avoid conflict with the off-bound traffic from the freeway.

In many cases, frontage streets are not necessary. Adjacent property can be laid out to back up to the expressway right of way or to face on streets running at right angles

to the expressway. This may result in dead-end or cul-de-sac streets, which, for residential purposes, may be desirable.

In closing, I wish to point out that the design of an urban expressway is affected considerably by the existing and proposed development of the area through which it passes. Design standards must be flexible enough to permit variations to meet these conditions.

REFERENCES AND EQUATIONS

(1) A Basis for Estimating Traffic Diversion to New Highways in Urban Areas
A.A.S.H.O. Subcommittee on Factual Surveys, December, 1952

(2) Highway Capacity Manual — Page 145

(3) A Policy on Geometric Design of Rural Highways — Page II - 36

(4) A Policy on Geometric Design of Rural Highways — Figure II - 8

(5) A Policy on Geometric Design of Rural Highways — Page II - 42

(6) A Policy on Geometric Design of Rural Highways — Page II - 50

(7) Studies of Weaving & Merging Traffic, Yale University — Pages 57 and 100

(8) Highway Traffic Analyses — Page 195

The probability of any driver having to wait for n intervals each less than t seconds is:

$$(1-e^{-Nt})^n \cdot e^{-Nt}$$

where e is the base of Naperian logarithms = 2.72,
and N equals the number of vehicles per second

(9) Highway Traffic Analyses — Page 194

The average length of all intervals less than t seconds is equal to:

$$\frac{1}{N} - \frac{te^{-Nt}}{1-e^{-Nt}}$$

(10) Time required for gap to reach original position of entering driver

$$= 4 \times 0.9 + \frac{\text{Car length}}{\text{Freeway speed}} = 3.6 + \frac{20}{1.47 \times 40} = 3.94 \text{ sec.}$$

Distance traveled by entering driver before gap overtakes him is equal to:

$$\frac{\text{Freeway speed} \times 3.94}{\frac{\text{Freeway speed}}{\text{Av. speed of entering driver}} - 1} = \frac{1.47 \times 40 \times 3.94}{\frac{40}{27.5} - 1} = 510 \text{ feet}$$

CONSTRUCTION OPERATIONS SECTION

Afternoon: Room 3-D

Chairman: P. V. Pennybacker, Special Bridge Engineer
Texas Highway Department
Austin, Texas

1:30 Principles of Structural Welding

3:00 Inspection of Structural Welds
William H. Greer, Assistant Manager
Southwestern Laboratories
Houston, Texas

THURSDAY, SEPTEMBER 9

CONSTRUCTION OPERATION SECTION

Morning: Room 2-D

Chairman: R. L. Rothe, District Construction Engineer
Texas Highway Department
Lufkin, Texas

9:00 Use of Traveling Crushers to Reduce Oversize Pit Material
R. E. Stotzer Jr., Resident Engineer
Texas Highway Department
New Braunfels, Texas

10:30 Joint Seals for Concrete Pipe Culverts
W. H. Garrett, Senior Resident Engineer
Texas Highway Department
Littlefield, Texas

Welding Inspection

William H. Greer, Assistant Manager
Southwestern Laboratories
Houston, Texas

Welding has become an extremely important phase of our industrial way of life, and more uses for it are constantly being developed. The nature of many of these applications has made it necessary that special techniques and special equipment be employed, but much work continues to be done with the manual arc using coated electrodes.

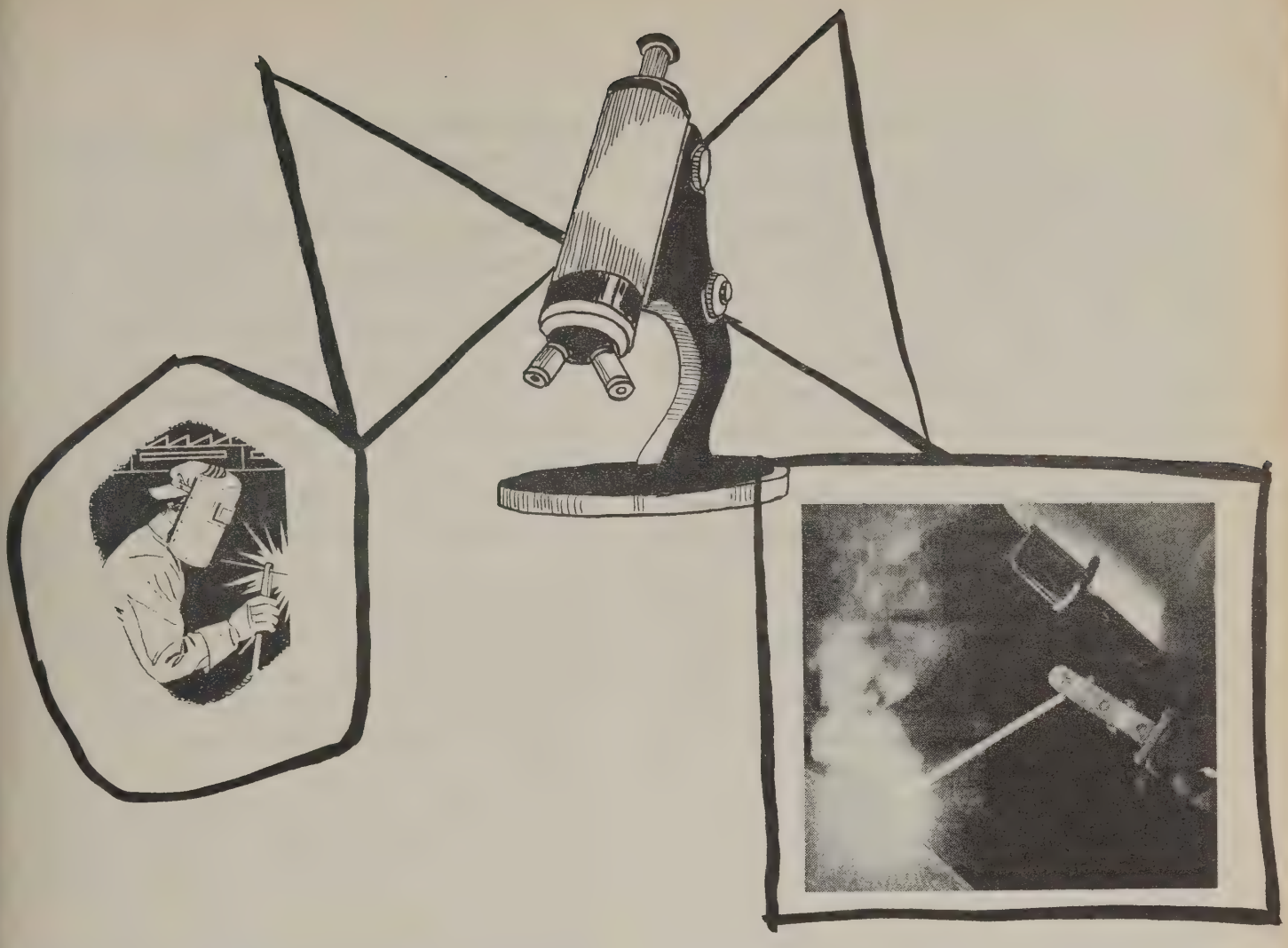
Each process requires certain standards for quality control in testing the welding procedures and the welding operators. In the case of weldments such as we are concerned with here today, the fundamentals have been established by the American Welding Society, and that organization remains on the job constantly, making investigations and doing research work. Those of us who are closely associated with American Welding Society activities know what an outstanding job it is doing.

First of all, permit me to quote from a section of the American Welding Society Standard Specifications for Welded Highway and Railway Bridges:

"Responsibility for Inspection: The Engineer responsible for constructing a bridge, the integrity and capacity of which will depend upon welding, is charged with full and final responsibility for the welding.

"Inasmuch as this responsible Engineer frequently will not be a qualified operator of welding equipment or a welding inspector, nor, however qualified, engaged personally in the act of inspection, he should discharge his responsibility in the following ways.

"He should assure himself of the experi-



ence, equipment, and adequacy of his welding contractor — assuming the work to be let by contract — or of his operators and supervisors, if the work is to be done by forces under his direction.

"He should assure himself of the experience, competency, energy and firm purpose of his chief inspector and of assistant inspectors, if any.

"He should provide for adequate and independent inspection of welding to be done by his own forces, and this inspection must be as independent of the producing force as if the work had been let to a contractor.

"He should assure himself that harmony and a common purpose exist between his contractor, or the foreman of his own welding force, and his welding inspector.

"He should establish and insist upon a system of records that will allocate and record a definite and permanent responsibility for the acceptance of every weld as to size, location and quality, and the certainty that no weld called for by the drawings has been omitted.

"Inasmuch as there is no visual test, and no other type of test which can ordinarily be applied at reasonable cost, by which to determine absolutely the degree of perfection of a structural weld from its final appearance alone, or by hammering or otherwise operating upon it (short of destruction or severe injury), inspection of welding resolves itself principally into:

(a) Qualification of welding operators
(b) Inspection before and during welding."

Let us consider now some of the fundamentals that govern testing of welding procedures and operators for this class of work.

Procedure qualification involves such items as:

1. Filler metal specifications
2. Electrode classification numbers
3. An increase in electrode diameter
4. Position in which welding is done
5. A change in shape of the welding groove
   a. Decrease in included angle
   b. A decrease in root opening
   c. Increase in root face

d. Addition or omission of a backing strip

To qualify the procedure, a suitable plate assembly must be made with the characteristic joint details, and this procedure should be tested in each position to be encountered in the work. From each test-plate weldment, we obtain (a) two reduced section tensile tests, (b) two free-bend tests for ductility, and (c) two root bends and two face bends for soundness (or four side bends if the material to be welded is one inch thick). The operator making a satisfactory procedure test is likewise qualified for that position.

Inasmuch as your Bridge Division establishes the physical aspects of the joint design, and has evidence of its suitability when electrodes of a given size and classification are used, your main consideration may be with the contractor's equipment and personnel. Inspection of his welding machines, cables, and accessory equipment will frequently indicate its general condition. Gasoline-engine-driven generators should respond quickly when electrode contact is made, and should reflect ample power for the work load. Grounding cables to the structure should be large and have intimate contact with bare metal surfaces. Grounding cables will usually be warm to the touch but should never be hot. Such a condition indicates a poor cable and/or connection.

Leads to the electrode holder should be in good condition with tight connections and a minimum of splices. They should be kept as short as practicable.

The operator must have steel wire brushes and welder's hammers as a minimum of equipment for cleaning weld beads as the joint is made. Pneumatic scaling tools are preferable but are frequently not so readily available under field conditions.

When the procedure has been established and proven satisfactory, welding operators must be tested in accordance with standard procedures. This involves their welding of prepared plates in positions necessary for the particular project, and will include separate tests for fillet welding and butt welding. From fillet weld test plates, two bend specimens are required and both are tested with the root weld bead on the tension side of the bend. From butt weld test plates two bend specimens are required, one made with the root of the joint on the tension side, and one with the top or face of the weld on the tension side. An operator qualified in the vertical and/or overhead positions for fillet welding is not required to test in the flat or horizontal positions. An operator that is qualified in the horizontal or vertical or overhead position for butt welds is not required to test in the flat position.

In conducting fillet weld operator tests the joint assembly is kept in the test position only long enough to complete the root beads. The plate is then taken down and the weldment completed in the flat position. In conducting butt weld operator tests the joint assembly is kept in the test position until the weldment is complete.

During tests on small plates heat builds up rapidly and some adjustment of machine settings may be necessary after the first few passes.

Test specimens should be carefully prepared with reasonably smooth edges, and with weld reinforcement removed flush with the plate. Sharp corners on the tension side should be rounded, and the weld should show good penetration, soundness, and fusion.

Qualification records should be maintained on each operator.

When components of the structure have been assembled, aligned, and adjusted, it is imperative that the joint assembly show proper spacing, with the web and flange elements in good alignment. The joint spacing should not be less than shown on the plans because during initial stages of the welding, shrinkage in the metal will tend to draw the parts together. If this occurs without prior allowance for it, the root of the joint may be so closed as to prevent proper penetration. Adequate joint spacing is a prime requisite for welding.

With welding under way in accordance with the approved procedure and sequence the inspector concerns himself with the nature of the deposited beads. The operator must so adjust the machine setting and his manipulation as to produce penetration and fusion of

the parts with a uniformly smooth deposit showing good soundness with no overlap or undercut. Each bead must be thoroughly cleaned of slag and spatter metal before the next bead is put on, and any severe inclusion, fracture, pocket, or lack of fusion should be gouged out to permit proper welding from the bottom of the defect. Gouging can be done with pneumatic chipping equipment, torch gouges, or disc grinders. For such repairs, and on the initial pass in any joint, the electrode size should be small. The finish bead should be uniform with not much reinforcement.

Since temperature changes cause a change in volume, we are faced with the problem of shrinkage due to welding. More complicated structures induce additional shrinkage stresses, but the condition prevailing on bridge work is usually not too severe. Investigation of the effect of heat on steel reveals some very interesting facts. A bar one inch long at 75° F. increases in length when the temperature is raised 1° F. The compressive force required to return the bar to its original length would be approximately 200 p.s.i. This figure has to be qualified because the coefficient of expansion and Young's Modulus change with temperature. If, however, we use the 200 p.s.i. as approximate for 1° F., it is readily apparent that a temperature change of 1700° F. has terrific effect. The compressive stress for such a change would be a theoretical 325,000 p.s.i.

There are two reasons why we never reach such values when a bar is restrained during heating. First, the yield strength of the material is reached at about 40,000 p.s.i., or after a temperature increase of 200° F. If we continue to increase the temperature, we get continuing deformation in the steel and/or movement of the structure. The second reason our values fall short is that the yield strength of steel decreases, until at about 1000° F. it vanishes altogether. As our temperature increases due to welding, the steel tries to elongate, and if it is restrained we get an upset condition. Then as the metal cools it cannot return to its normal condition because of the upset near the heat zone. When molten metal is placed in a prepared joint, it shrinks when cooled, and our problem is further complicated. Stresses with which we are primarily concerned will then become tensile stresses across the joint. If the components are rigidly restrained, the stress will be equal to the yield strength of the material and/or weld deposit. In most instances the yield strength of the weld deposit is higher than that of the parent metal, and this feature tends to prevent high concentration of stress in the weld alone.

Two other procedures are recommended as aids in welding heavy sections under high degrees of restraint. One of the most effective of these is preliminary heating of the joint area to a temperature between 400 and 650° F. just prior to welding. This procedure prevents quench cooling, and permits a slower and more uniform cooling of the entire joint area with a consequent improvement in stress distribution. Preheating should be seriously considered when atmospheric temperatures are below 50° F., and when the job to be done involves heavy sections in a high degree of restraint. A second method for reducing shrinkage and distortion is accomplished by peening weld beads with a blunt tool after each pass is made. This should not be done on the first pass because it may cause a fracture that will progress through the weld, but each succeeding bead may be peened sufficiently to cause slight upset of the deposited metal. It is not necessary to peen the "finish" pass, and the interest in good workmanship recommends against it.

Fillet welds create angular distortion, and direct shrinkage is much less apparent where that type of joint is employed.

Distortion and shrinkage are influenced by the rate of heat input, and this is related to the electrode size, machine setting, and number of beads made in the joint. The more beads made in a butt weld, the more stresses and attendant distortion we expect. This makes it appear that large electrodes should be used wherever possible, but there are other influences to be reckoned with. Near the bottom of a beveled joint the opening is narrow, and, since our electrade must be fairly close to the fusion zone for creation

of a suitable arc, we need a smaller diameter electrode. The operator also needs the advantage of having a smaller quantity of molten metal with which to contend during the first beads, and this feature is even more important in position welding. A fairly large percentage of welding operators are unable to make satisfactory welds in a vertical position using 3/16-inch diameter electrodes. The percentage improves when we reduce the diameter to 5/32 inch and is even better when the rod size is 1/8 inch. Then the inspector and/or engineer will establish a compromise technique in favor of good weld soundness and fusion.

Mr. Joe Lawrence Morris, writing in "Welding Principles for Engineers," has said:

"The great majority of all welds made are passed or approved by visual examination. This test is far from infallible, but if the general appearance of the finished weld meets certain standards, the product is usually considered to be satisfactory, except in those cases where danger to life and property or other serious losses may be incurred by failure of a welded joint."

This observation is correct, but in my opinion, it does not and must not apply to the class of inspection with which you are concerned. Most other codes covering important work require that the inspector see the fabrication during the assembly and welding of the various components as well as after all work is complete.

You will not be able to see every phase of work performed by every operator on the job, but you should observe some of his welding. Check the joint assembly before he starts; be sure there is ample root opening and proper alignment of parts; be sure electrodes are fresh stock and free of excess moisture; note the manner in which he holds the electrode — he should incline the rod ahead of the puddle, and, since the arc force is directional, heat must be directed adequately to all parts being joined. The arc should be steady, and manipulation should be deliberate and so controlled as to avoid undercut sides or highly irregular bead surfaces. Before the first welding is done on the back side of the joint, check the condition of the assembly. Be sure there is ample clearance to fully penetrate the original root bead. Rough areas, excess metal, voids, and narrow spacing should be gouged or ground to provide proper conditions for welding. Check the operator's welding sequence and progression as the joint is being welded; be sure corners and edges are built up to full section, and that finish beads are uniform. Be sure each operator identifies his work for record and reference.

The welding codes do not make it mandatory that the inspector witness the making of operator qualification test weldments, but the inspector must satisfy himself as to the ability of each welder, and be prepared to ask for a retest if production welds become questionable. The physical and mental condition of an operator very often affects his work, and a "welding lay-off" of three months or more may recommend requalification. In all phases of this inspection you will be required to render an opinion, and your judgment will not coincide exactly with that of a colleague. Borderline cases of acceptability will make it necessary that you use good judgment and consider all aspects of the case in question.

There are several ways in which a completed weldment can be examined by random sampling. Trepan plugs are samples cut through the weldment by a hole saw, and they offer evidence as to density, penetration, and fusion. Radiographing is a nondestructive shadow picture of the weldment that will show defects having a depth magnitude of not less than 2 per cent of the metal section. Magnaflux inspection is a means of checking for surface defects, but the effective penetration is usually not more than 1/8 inch below the metal surface. Ultrasonic inspection can show defects of any magnitude under ideal conditions and with a properly experienced technician. All of these methods have merit, especially with respect to the psychological effect on the welding operators, but they are comparatively expensive. In general, good quality work can be obtained by steady attention to fundamentals of the welding operation after proper qualification of the procedures and operators.

THE PORTABLE ROCK BUSTER

R.E. Stotzer Jr., Resident Engineer
Texas Highway Department
New Braunfels, Texas

When first asked to make a talk on the portable rock buster here at the Short Course, I became somewhat panicky at the thought of talking about any one thing, other than those "fighting Aggies" for the length of time they want this program to last. So immediately I got on the telephone and called Mr. Browning of the Browning Manufacturing Company, the people who make and sell the rock buster, and asked him if he had any ideas as to how I could carry out my program. He volunteered the use of a movie film and some slides and that eased my task considerably.

My introduction to this machine came some ten months ago on F. M. 306 in Comal County. On this particular project a pit had been set up for use as unprocessed foundation course and, as often happens, a considerable amount of oversize material was found to be present after construction was started. On a large portion of the road, as much as fifteen to twenty cubic yards of the 50 cubic yards per station dumped was oversized and very hard. This oversize material was made up of conglomerate and limestone. Samples of this rock are here by my side and you can see them at the conclusion of the program. A D-7 tractor and sheepsfoot roller would not break down more than a small portion of this.

The contractor on this project was Colglazier and Hoff, of San Antonio. After a couple of days of trying to walk this material down with a tractor, it was obvious to both the contractor and our highway personnel that something else would have to be tried. I had heard about this "rock buster machine" being demonstrated on a county road near San Antonio and so I mentioned it to C. L. Ferguson, Superintendent for Colglazier and Hoff, who immediately went to work on Jim and Bill Colglazier to try and get the machine to demonstrate what it could do in some really tough going on our job.

Within a day or two arrangements had been made to bring the machine on the job and we proceeded to see what it could do. By blading out the oversize material into several windrows and pulling the rock buster over it, it was possible to make use of all the material hauled onto the road. (Incidentally they used a D-7 to pull it, but it doesn't necessarily require that much tractor.) Ordinarily going over a windrow twice was enough, but occasionally due to the size of the windrow it was necessary to go over it three or four times. Actually, the harder the oversize was, the better it seemed to break because the steel hammers would shatter it more.

The rock buster is a multiple hammer-type machine equipped with eighteen 36-pound hammers rotating at the rate of 1000 R.P.M. and powered by a 90 H.P. diesel engine. Its over-all width is eight feet and its breaking width is four feet. It is sixteen feet five inches long, nine feet high, and weighs about 12,000 pounds. The average life of the hammers varies from 150 to 600 hours depending upon the abrasiveness of the material processed. On this particular project the machine was in use 52 hours on the four and one-half miles of base material. At the conclusion of the job we felt that this time could have been reduced by 20 per cent through the know-how gained in using the machine. The machine was also used on a short section of windrowed boulders which had been previously bladed out of the roadway and against the R.O.W. fence. Here again the machine showed another desirability, that of ridding the R.O.W. of unsightly boulders and leaving it clear for thorough maintenance.

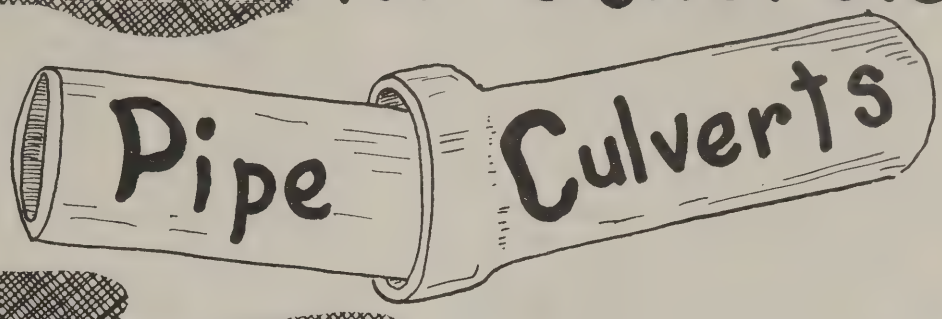
By the use of this machine it seems quite probable our riding surfaces will be improved and road costs greatly reduced. In many instances it would be possible to reduce haul distances by the use of material sources that would not ordinarily be used because of the amount of oversize material in the pit and the prohibitive cost of using a crusher on small quantities. I know that we currently have a project under contract in Comal County where we were able to use a pit that, without the knowledge that we could revert to the rock buster, the oversize would have scared us off and forced us to haul a much greater distance.

On this project we specified a maximum size of two and one-half inches on the foundation course and emphasized to the prospective bidders while showing the job that all acceptable material in the pit would be used. They were told that it could be processed either in the pit or on the road, thus leaving the bidder to decide which would be the most economical way to meet the specification.

In the past our practice has been to break down the oversize base material as much as possible with a tractor and sheepsfoot roller, then blade the remainder of the oversize material into the ditch to be buried or windrowed against the fence. In some cases this has amounted to considerable yardage; however, the big objection in my opinion has been the pitted surface that sometimes develops after the completion of the project caused by the oversize material in the thin course of base. The quality of the material after processing it with the rock buster approached that of base material run through a crushing plant.

The rock buster costs approximately $10,500 and currently rents for $1,000 per month. Using the rate of processing that we got on the Colglazier and Hoff project, the additional cost of processing would be about 6 or 7 cents per cubic yard. Processing this same material with a regular crushing plant would probably cost us somewhere between 35 and 45 cents per cubic yard.

Joint Seals for Concrete Pipe Culverts



W.H. Garrett, Senior Resident Engineer
Texas Highway Department
Littlefield, Texas

I have had the opportunity of attending several Highway Short Courses and have gained a great deal of information and benefit from the excellent papers delivered and the discussions that have come out of the meetings. I have prepared this paper with the thought in mind of having a great deal of discussion and I hope you will join in a discussion of this subject. I think an open discussion will give many of you an opportunity to express your ideas and opinions on any particular point.

For after all, new ideas and new solutions to old problems are the lifeblood of the engineering and construction professions. Engineers and contractors alike must be open-minded and receptive to suggestions for new methods, equipment, and materials. Many procedures and materials that were unheard of, or at least thought to be farfetched, ten or fifteen years ago are now in common use.

So it is with concrete pipe for use in drainage structures.

Concrete pipe is not a new product. It has a long and enviable record of service in the culvert field for highways and railways. Concrete pipe culverts were developed by the railway engineers and the highway engineers to fill a need for a rugged permanent culvert pipe. Two railways have used these culverts for 80 years. The state of Iowa has used this type culvert for highway structures since 1906, and the universally accepted life expectancy of one is 100 years. The lack of manufacturing facilities in many areas has limited its use for highway drainage structures.

In the past 10 years there has been a marked increase in the use of this culvert, both in the Land Service Road Program and in the primary highway system, which is due primarily to two factors: the increased manufacturing facilities, and the economic advantages.

In my area there is extensive use of concrete pipe for underground transportation of irrigation water, so the demand has placed a manufacturing plant within reasonable shipping distance of most highway projects.

Since the increased demand for low-cost highways has brought about the extensive use of this product for culverts, engineers and contractors are faced with the problem of choosing a method of installation that will be economical, and at the same time be adequate for existing conditions.

So if you are going to use concrete pipe, you will have to make the joints, and it is going to be up to you to choose the type of joint. I will give you my ideas on the subject, and will welcome your questions or comments at any time.

First, I will take up the laying of the pipe, since in my opinion the first requirement for a good, well-fitting joint is a uniform, well-prepared, properly shaped trench or foundation in which to place the pipe. The location of the pipe is staked out for center line of roadway and center line of pipe

culvert, and the flow line elevation of pipe given.

Then the excavation and fine grading of the trench is completed. The bottom section of the trench is shaped to fit the lower part of the pipe exterior for approximately 10 per cent of its over-all height. This lower part of the excavation must be cut to the shape of the lower part of the pipe and must be uniform throughout the length of the culvert, or a poor joint connection will be the result. A template cut to the shape of the outside of the pipe must be used in checking the shaping of the bedding.

The laying of the pipe shall start at one end of the culvert and proceed to the other end. In my opinion it makes no particular difference where one begins, but, usually the outlet end is preferred. The tongue end of the pipe shall point in the direction of flow.

There are four main types of joints for gravity flow concrete pipe culvert and sewer pipe: the bell and spigot or bell end type joint, the modified tongue and groove type, the tongue and groove type, and the concrete collar type of joint.

The tongue and groove type joint is the one most commonly used in pipe culverts and I will first talk about the construction of this type of joint. The first pipe downstream should be bedded to establish line and grade, with the groove upstream. The groove should be carefully washed with a wet brush and the bottom half of the groove end filled with mortar (all mortar shall consist of one part portland cement and two parts sand properly graded). The tongue of the next section of pipe is cleaned with a wet brush and a layer of mortar is applied to the top half. The tongue end of the pipe is then fitted into the groove end until the mortar is squeezed out onto the inner and outer surfaces. The inner surface of the pipe at the joint is then brushed smooth and the outside pointed up with a bead of mortar approximately two inches wide and one-fourth to one-half inch high to complete the joint seal. The mortaring of these joints is purely a hand job, supplemented with trowel and brush. The workmen should wear some type of glove for protection of the hands. This, the cement mortar joint seal, is probably the oldest jointing material used. The secret of its success is adequate inspection to insure that it is properly made. It is a watertight, rigid joint widely used with complete success.

There are several jointing compounds on the market for use with the bell and spigot, tongue and groove, or the modified tongue and groove joint. Some of these jointing compounds have been used with complete success and the use of others has resulted in disastrous failures.

Large cities like New Orleans and Los Angeles and Los Angeles County report success with jointing compounds. I have never experienced the use of any type of these compounds and do not have any further information on them.

There is another type of joint seal that I would like to discuss a little. That is the cement mortar oakum joint.

In this type of joint the tongue and groove pipe is laid in the same manner as previously discussed with the exception that oakum is used in the joint in addition to the cement mortar. In pipe sizes 36 inches and over the caulking is done from the inside and the tongue is generally made shorter than the groove to provide space to accommodate the oakum. After the tongue end of the pipe is fitted onto the groove end of the other pipe, the mortar has to be cleaned out, making space for the oakum. The oakum shall have been previously dipped in a mortar mix of one part cement and two parts sand. The oakum is then placed in the joint with a caulking tool. The inner surface of the joint is brushed smooth. Oakum is a closely twisted hemp or jute gasket.

In pipe sizes under 36 inches the caulking is done from the outside and the tongue is made longer than the groove to provide space for the oakum. After the oakum is caulked into the joint, it is covered with cement mortar to form a bead over the joint.

This is the type of joint I would suggest for use in concrete culvert pipe.

I would also like to mention the single rubber ring gasket joint which is widely and successfully used on concrete pipe sanitary sewer lines and might be used successfully on concrete culvert pipe. This rubber ring is used more often with the tongue and groove type joint. It provides an excellent watertight joint. It is completely root tight, and yet has a certain small amount of flexibility -- enough to allow for slight settlement movement in the line. This single ring gasket is similar in principle to the rubber rings used on fruit jars with added shapes and corrugations so that the joints slip together easily and come apart with difficulty. The rubber ring gasket is slipped over the tongue before the section of pipe is laid in the trench.

The tongue of the pipe with the rubber ring on it is then inserted into the groove end of the pipe in place in the trench, thus sealing the joint. However, the pipe must be manufactured to close tolerances so that there is the correct space left for the single rubber ring.

Anyone wishing more information regarding the dimensions for the pipe ends, tolerances, and instructions for installation may get such information from the manufacturers, the names of which will be furnished upon request addressed to the American Concrete Pipe Association, 228 North LaSalle Street, Chicago 1, Illinois.

There has been some discussion as to how long these mortar joints should be cured or left open before the backfilling is done. The green book of the Texas Highway Department Specifications calls for the joint material to remain open for 24 hours, but this has been amended by Special Provision calling for six hours minimum time before the backfilling is done. I can see no particular reason why the mortar joint cannot be backfilled immediately, while the mortar is still plastic. However, the mortar joint must be covered immediately with wet paper, wet burlap, or other wet material for proper curing and protection. The paper from cement bags makes a very satisfactory and economical cover.

The joints of the irrigation pipe in my area are made by filling the groove completely with mortar and forcing the tongue end into the mortared groove and then sealing the joint inside and out practically the same way as previously stated for the pipe culvert joint. The backfilling is done immediately. This method of sealing the joint of underground concrete pipe is used by a local company and may not be the universal method of joint sealing.

In some cases, where traffic would be inconvenienced, I have had the backfill placed immediately after the joints were sealed, using a wet burlap material in the particular case for cover over the mortar joint, with very satisfactory results.

I have laid concrete pipe culvert with no mortar in the groove -- what I would call a dry joint -- simply laying and fitting the ends of pipe as closely as possible and then sealing the joint inside and out with mortar in the same manner as previously stated. This jointing has proven to be satisfactory. I believe it will allow the pipe to be salvaged more easily if it is necessary to remove the pipe in the future. The mortar bondage in the groove would cause the tongue and groove to be broken in removing the pipe.

Now the question may arise as to whether or not it is necessary to use a joint seal at all. In some cases I think I would lay the pipe with a dry joint, without sealing at all, due to the type of backfill material. If there is a seepage or free-water collection around the pipe, the dry or unsealed joint might be advisable since this type of jointing would afford a means of bleeding or drainage for the backfill around the pipe. I have been told that there are some engineers who have the pipe laid with the joints unsealed, and if there is anyone here that would give us some discussion on this type of joint, I would like to hear from you.

There is one thing you must bear in mind, and that is, whatever type of joint you may choose, it takes an experienced workman to make the successful joint, sealed or unsealed.

MAINTENANCE OPERATIONS SECTION

Afternoon: Room 2-B

Chairman: J. W. Cravens, District Maintenance Engineer
Texas Highway Department
Paris, Texas

1:30 Controlling Encroachments on Highway Right of Way
O. L. Crain, Assistant District Engineer
Texas Highway Department
Bryan, Texas

3:00 Shoulder Surfacing as a Maintenance Operation
I. H. Crutcher Jr., District Maintenance Engineer
Texas Highway Department
Tyler, Texas

Grover Green, District Maintenance Engineer
Texas Highway Department
Dallas, Texas

CONTROLLING ENCROACHMENTS ON HIGHWAY RIGHT OF WAY

O.L. Crain, Assistant District Engineer
Texas Highway Department
Bryan, Texas

It is with considerable pleasure that I am allowed to bring you a discussion on a subject about which I know absolutely nothing. You who hear me will feel as the father did who was listening to his son expound on what he would do when he became a father. The father looked at his son and told him that he had better "do all his talking now as he would find that he knew less and less when he actually became a father." Having spent the first 22 years of my career in the construction end of the Department and only a short time in active maintenance work, I am definitely in the position of the expectant father, insofar as encroachments are concerned.

When Fred Benson asked the districts for subjects for discussion for this Short Course I suggested the subject of encroachments, as it appeared to us here in District 17 to be getting worse. One writer from Whittier, California, seems to agree with us. In "Over the Highways" in the September issue of Texas Highways a tourist stated - "Did not like our lack of control of right of way." On the other hand another tourist was impressed by the lack of advertising signs. This indicates extreme variation in the problem under discussion.

In the past four years I have traveled in fourteen districts, other than District 17, and have talked to a number of maintenance men and it appears that encroachments are becoming more of a burden each passing year. Mr. Greer, in a Maintenance Order dated February 26, 1946, says "- - - more and more unauthorized signs are appearing on our highway right-of-ways. Although in the strict sense of the word the majority of these may not be of a commercial advertising nature, they are probably detracting the attention of the traveling public from our highway markers.

"At a time when traffic accidents are on the increase, we feel that a renewed effort should be made and all field employees be instructed again that no unauthorized signs be permitted to remain within the limits of the right-of-way. - - -"

When Fred asked if I would take this subject, I agreed only with the understanding that I could call on others in the Maintenance Division for discussions and with audience participation we all could learn as much as possible about controlling encroachments on our highway rights of way.

Before I call on anyone for comments, I would like to give you a brief background of the laws, department policies, and definitions, which form the basis of departmental handling of encroachments.

The first and main law under which we operate is Article 784 Texas Penal Code 1925.

Second - Circular Letters - 1936 - 1939 and 1946.

Third - Maintenance Manual specifically refers to removal of signs, posters, etc.

Land for our present highway system was acquired in four ways. In fee simple, easement, prescription, and consent. The first method - fee simple, means title without restrictions or limitations. Easements are granted for a specific purpose, and are limited to the life of the purpose for which it was granted. Prescription is the establishment of a claim of title by use and enjoyment during a time fixed by law (not title to land, etc., except for public purposes or use). Consent - whereby we ask and get permission to do certain things. This can and does run into the same right as given above by prescription, except that by prescription we may or may not have gotten previous permission.

The Department as we know it today acquires land for highway purposes by warranty deed, that is in fee simple or by easement. Many of our old highways as well as many of our new farm-to-market roads exist on rights of way of unknown origin. Like Topsy, "They just growed." However, we today are concerned not as much with how we obtained our rights of way as we are with how to keep them clear of encroachments, although the method of acquiring rights of way does have a bearing on the ease with which we can clear the encroachments.

Webster's Dictionary defines encroach as "To enter by gradual step or by stealth into the possessions or rights of another;to trespass;intrude." So, any unauthorized signs, plowing, planting, fencing, buildings, pipe lines, utility lines, selling of potatoes, beans, watermelons, used cars, cattle, dumping of debris and dead animals to us constitutes an encroachment or trespassing on our rights of way. The highways are for use by the traveling public and they have every right to expect that they can travel them with safety, minimum of cost, and maximum of pleasure.

An unauthorized encroachment constitutes a hazard, is an unnecessary waste of funds to remove, and in most cases does not afford pleasure to travelers. The State Highway Department is a service organization, tax supported by the operators of vehicles over our highways and certainly we, as employees of the Department, should not be arbitrary, rude or impatient with the individual violator. However, when the individual does encroach on the rights of the traveling public we must take a positive stand to prevent and/or remove the encroachment to insure traffic safety and preserve the utility of our highways. The best defense against encroachment is a fast and efficient offense. That is, constant vigilance, prompt action, and persuasive arguments by all personnel of the Department. Resident Engineers and their forces should be just as interested in maintaining clear and unobstructed rights of way as are the Maintenance Foremen and other maintenance personnel -- not necessarily to take positive action but to be on the alert and report and resist any and all encroachments. It is every one's Department and we all should be equally proud and eager to keep it the best Highway Department in these United States.

As I told in the beginning, my experience in maintenance work is limited, but not my opinions. They are varied and unlimited, subject to change with very little warning, and I give them to you now for what they might be worth.

I believe the key to control of encroachments, assuming of course that you have some, lies with our Maintenance Foremen and their county forces. Of course, they must have the backing and support of their superiors as well as the cooperation of the county courts and law-enforcing agencies, but they remain the key figures. They are required to cover all their highways every week, with coverage of important highways every day. They should see the encroachment, take prompt action, and if this action is ineffective, report to their immediate supervisor and let him carry the ball. Now enters a peculiar trait common to all of us. Seeing or riding over a highway innumerable times tends to dull our recording mechanism to the point that our brain does not record what our eyes see. Seriously, at a recent meeting of our Maintenance Foremen, we discussed this very problem and debated the value of the foremen exchanging counties for one or two days each month in order to improve the general quality and degree of maintenance of our highways. But, back to the foremen seeing the encroachment and not being able to remove or have it removed, the supervisor is handed the ball. I have one ball which was handed to me a week or two ago. Just recently the district passed this same ball to the county court of one of our counties. So far they have not fumbled. If for some reason the cooperation and assistance of the county court cannot be obtained, we still have the right to file charges against the encroacher.

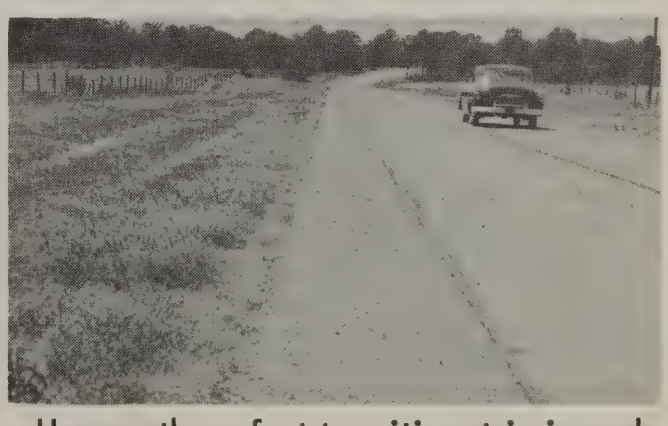
Shoulder Surfacing as a Maintenance Operation 1.

I.H. Crutcher Jr.
District Maintenance Engineer
Texas Highway Department
Tyler, Texas

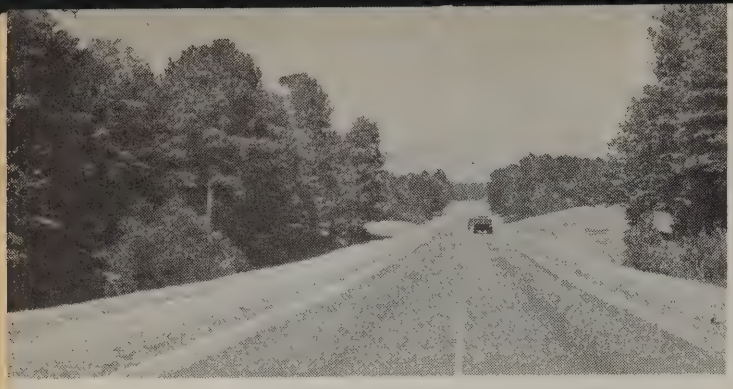
With the permission of the Chair, I shall modify the title of the assigned subject to "Partial Shoulder Surfacing as a Maintenance Operation."

Iron ore gravel, our only local base material, is unsatisfactory for use on shoulders without an asphalt surface. The rapidity with which it blows away in dry weather, combined with increasingly long hauls due to fast dwindling sources, makes replacement costs prohibitive. An asphalt surfaced shoulder, wide enough to encourage its use as an extra traffic lane is subject to early failure unless the base is of adequate thickness to withstand heavy wheel loads. We have a considerable mileage of shoulders of this kind, usually placed as a construction item when the road was paved. The high cost of this type shoulder quite obviously precludes its use as a maintenance operation on any considerable scale. In areas near cities, or in the vicinity of extensive commercial or residential development along the highway, the full width surfaced shoulder may be essential. But in strictly rural areas, even if sufficient funds were available, we question the necessity of such an investment in our locality.

We consider Bermuda grass sod indispensable for shoulders in most conditions in our district. As a compromise measure, on our heavier traffic roads, we are now making extensive use of what we call transition strips between the pavement edge and the sodded outer portion of the shoulder. This procedure originated actually as pavement widening. Many of our principal highways were paved 20 to 30 years ago when eighteen or twenty or even sixteen feet was the usual width. As traffic increased, proper shoulder maintenance became virtually impossible. To meet this problem, we started many years ago a consistent program of pavement widening. The procedure is to cut a trench nine to ten inches deep and two to four feet wide on each side of the pavement. We made a most satisfactory tool for this purpose by cutting a short section from an old moldboard and bolting it to the moldboard of the grader to be used. After scarifying

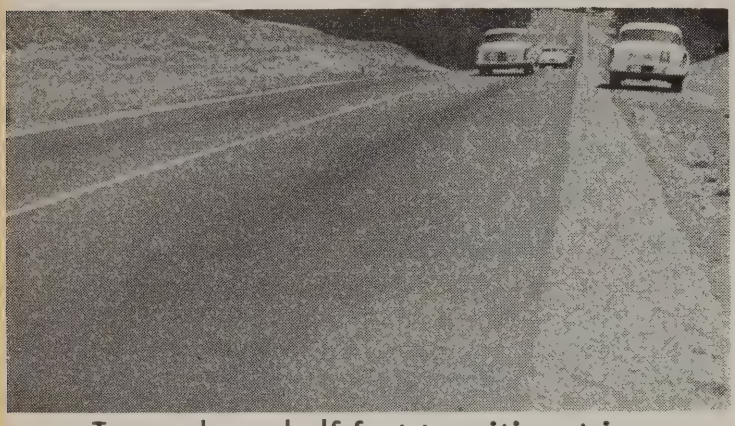


Here, a three-foot transition strip is used with eighteen-foot concrete pavement.

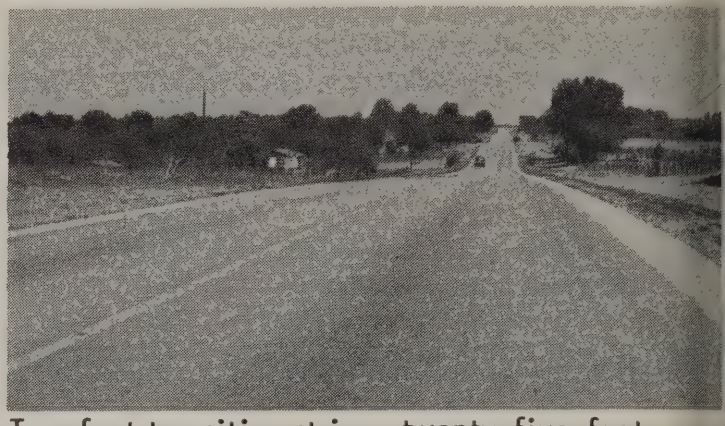


Two and one-half foot transition strip -- Twenty-four foot rock asphalt pavement.

Two-foot transition strip -- Twenty-two-foot concrete pavement.



Two and one-half foot transition strip -- Twenty-four foot hot mix asphaltic concrete pavement.



Two-foot transition strip, twenty-five-foot hot mix, cold laid asphaltic concrete pavement.

the shoulder adjacent to the pavement about six inches narrower than the desired width, this implement will cut a uniform trench with a regular outer edge. The trench is then filled with iron ore gravel. Compaction is secured by blading part of the material out of the trench onto the pavement, wetting, and rolling with truck and maintainer wheels as it is brought back in. It is then primed and a single asphalt surface applied. Since the color and surface texture of this widening strip differs from that of the original pavement, most traffic shows a tendency to avoid the widening strip except when crowded by opposing or passing vehicles. With very little traffic along the edges of the widened pavement, the maintenance of smooth sod shoulders is relatively easy.

Later, as it became desirable to resurface these roads to improve their riding quality or to strengthen them, they were covered with asphaltic concrete or rock asphalt carried out over the widening strips to provide a pavement width of 24 to 25 feet. Since the color and surface texture of the pavement was now the same full width, traffic tended to travel near the edges and some rutting of sod shoulders developed.

Taking our cue from the results obtained by widening strips along narrow pavements, we reasoned that transition strips of contrasting color along the wider pavements would be worth the investment as a safety factor and repay their cost through reduced shoulder maintenance. The usual width of these strips is one and one-half or two feet, though we have built some as narrow as one foot and some as wide as five feet. Costs, of course, are contingent on the length of haul for the base material. Our average is about $1,250 to $1,500 per mile for a two-foot width. For such a job a typical estimate (below) would read:

TABLE

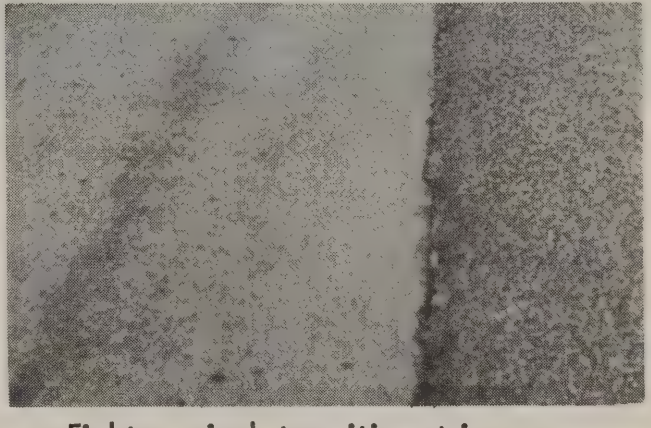
Iron ore base in place - 6 mi. avg. haul - 20 c.y. per sta.	@ $1.00 per c.y.
RC-2 prime - 25/100 gal. per s.y.	@ 0.14 per gal.
Native sand - 1 c.y. to 140 s.y.	@ 2.50 per c.y.
OA-135 - 25/100 gal. per s.y.	@ 0.13 per gal.
Aggregate, Type B, Grade 3, 1 c.y. to 80 s.y.	@ 6.00 per c.y.

In order to secure as much color contrast as possible we use crushed stone for aggregate, and use the minimum amount of asphalt required to hold the aggregate to prevent the surface from bleeding and turning dark in color. At night, these light-colored strips along the edges of a dark pavement are most effective in sharply defining the pavement edges.

The construction of these transition strips on our heavier traffic roads is now one of our major maintenance operations. And on our most recent flexible base jobs the base has been placed three feet wider than the surface. This will allow us to construct an eighteen-inch transition strip without the necessity of trenching.

I believe any discussion of shoulder surfacing would be incomplete without some mention of that eternal headache, the private access drive. We now have a pretty definite departmental policy requiring that surfacing of private drives be paid for by the owner. Unfortunately, 95 per cent of them will not or cannot do so. The deeply rutted shoulder and raveled pavement edge at many of these approaches constitutes a definite traffic hazard. Certainly it is our duty to our customers, the tax-paying motorists, to maintain the shoulders in a safe condition along the entire traveled roadway, including the approaches from private property. Smoothing up the rutted shoulders

Two-foot transition strip, twenty-four-foot rock asphalt pavement.

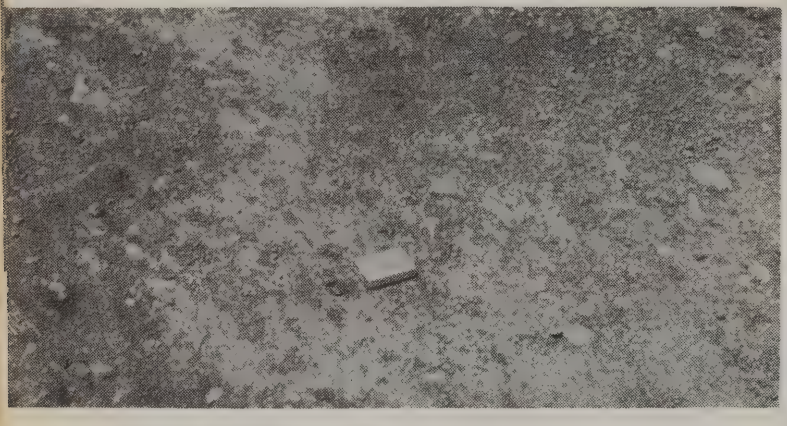


Eighteen-inch transition strip -- Twenty-two-foot concrete pavement.

Fifteen-inch transition strip, twenty-five-foot hot mix asphaltic concrete surface.

Bermuda-sod shoulder, twenty-four-foot hot mix asphaltic concrete pavement.

Oil mix surfaced private access approaches.



Close-up showing texture of oil mix surface.

Close-up of surface texture of transition strip.

with earth or even gravel at these points is usually a short-lived expedient, and frequent blading and addition of material are necessary for satisfactory maintenance.

Trying to solve this problem, we started experimenting about a year ago with cracked fuel oil, which costs us a fraction over 4 cents per gallon delivered. We mix this oil with blades, also using a Seaman Pulvi-mixer when available, with the material in the bottom of worked-out gravel pits, which usually contains a fair amount of gravel. Where we have no old gravel pits on reasonable hauls, we use the best available local soil, preferably sand clay. After mixing, the material is cast into windrows, where it will keep indefinitely, and is hauled out as needed. This has proved to make a satisfactory surfacing on the shoulders at private approaches. Actually we usually place the oil mix out to the culvert, where one is used, or across the ditch when access is by a dip. This may be stretching our policy a bit, but certainly the maintenance of the shoulder, road slope, and ditch is essential, and we assume it is not the intention of the policy to prevent maintenance of these critical areas.

In some of our early experiments, we made this mix too lean so that it tended to ravel and dust out, but by trial and error we can now produce a material which gives excellent results at access-drive approaches and mail box turnouts. From 25 to 30 gallons per cubic yard is usually required. Cost of the material in place will run around $3 per cubic yard. The amount used per square yard varies with the condition of the drive. On an existing gravel base we use a light cutback tack coat and place the oil mix one inch or less in thickness. When placed directly on earth we omit the tack coat and use two to three inches of the mix. Cost data on one job of 118 installations showed an average cost of $12.75 each. Another job of 124 installations averaged $14.30 each. We believe this procedure will help solve one of our most vexing problems, and that savings in maintenance will soon repay the cost.

Shoulder Surfacing as a Maintenance Operation 2.

Grover Green, District Maintenance Engineer
Texas Highway Department
Dallas, Texas

Proper maintenance of open gravel and rock shoulders on the principal highways in our district has been a matter of considerable concern to us for the reason that shoulders can be a serious hazard to safety unless kept flush with the pavement surface. On our heavier traveled highways, particularly the ones we still have of pavement width less than 24 feet, fast traffic and outside dual wheels of trucks and trailers whip and suck out the shoulder gravel or rock along the pavement edge, leaving a dangerous trench that can cause a driver to lose control of his vehicle if wheels leave the pavement. This continuing condition requires constant maintenance correction, either by addition of small amounts of gravel or crushed rock hauled to the shoulder at regular and frequent intervals or by blading shoulder material from the outside of the shoulder to the pavement edge. The latter is the practice most often followed and after a few bladings the shoulder begins to get noticeably low on the outside edge.

In an effort to provide greater safety and a higher standard of shoulder maintenance we embarked, in 1952, on a program of surfacing those principal highway shoulders which were constructed of a material that would hold a surface. The Trinity River gravel which is common in our district will not hold a surface with any degree of satisfaction so we have not surfaced shoulders constructed of this material. At the present time, however, we are treating some Trinity River gravel shoulders on U. S. 75 in Ellis and Navarro Counties by admixing binder material and will surface the shoulders if satisfactory compaction of the admixed gravel can be obtained. To date we have surfaced the shoulders on 133 miles of highways, 84 miles with maintenance forces and 49 miles by contract. An additional 32 miles will be surfaced before this month is over.

The surfacing serves the purpose of keeping the shoulder level with the pavement and, if the traveling public would use the surfaced shoulder as a shoulder, we believe that materially increased safety would be obtained and that an over-all reduction in shoulder maintenance cost would result. We have found, however, that immediately after shoulders are surfaced, a certain percentage of vehicles, predominately trucks, start traveling the shoulders thereby making a four-lane facility of the two-lane highway. As a general rule truckers are the most thoughtful and courteous drivers on the highways and a large percentage of them will pull out on the shoulders in order to let faster traffic pass. If the shoulder has a good, smooth surface the truck driver would rather ride the shoulder than to pull out periodically to let traffic pass, so he chooses the course of driving on the shoulder continuously. This practice has proven disastrous for us in that sections of six-inch depth gravel shoulders we have surfaced have suffered much damage as a result of heavy loads.

We experimented with the use of signs reading "Shoulders for Emergency Use Only" in an effort to keep traffic from using the shoulders as traffic lanes and believe that the signs brought about a fairly large degree of cooperation. It was not feasible, however, to erect signs which would have sufficient message to explain to the public why a nice surfaced shoulder should not be traveled on, and use of the signs was criticized by some and their use finally discontinued. It appears that unless we can devise some means of keeping traffic, particularly trucks, from using the shoulder as a travel lane we must construct all shoulders on principal highways to a depth sufficient to carry heavy loads. We are now following a practice of using a ten-inch minimum depth for shoulder construction on principal highways.

Another reason why the public should not use the shoulder for a traffic lane is that the bridges in most instances and culverts in some instances are not of sufficient width to allow uninterrupted travel on the shoulders. Some accidents have resulted from vehicles running into a bridge end when traveling on a surfaced shoulder in an attempt to pass another vehicle on the right.

Following are listed, as we see them, the advantages and disadvantages of surfaced shoulders:

ADVANTAGES:

1. Eliminate trench at pavement edge caused by tire suction on loose shoulder material.
2. Eliminate necessity of blading shoulders after each rain. We believe this increases safety by great decrease in number of times maintenance equipment is required on shoulders.
3. Present neater appearing section.
4. Move the point of damage from dry-weather cracks from near pavement edge to point near outer edge of surfaced shoulder.

DISADVANTAGES:

1. Tend to encourage use as travel lane and increase tendency of traffic to illegally pass on right.
2. Increase danger in use as an emergency parking area in that vehicle parking would be in more danger of being run into.
3. Increase cost of shoulders since greater thickness must be used to provide strength for carrying heavy traffic.

Our usual shoulder surface consists of 0.25 to 0.3 gallon per square yard prime coat of MC-1 and 0.3 gallon per square yard OA-135 or RC-2 surface shot covered with Type A, Grade 2 Aggregate at a rate of one cubic yard to from 80 to 90 square yards. The average cost of our shoulder surfacing performed by maintenance has been $1,114 per mile or 14 cents per square yard.

We have found that our best results have been obtained where we have had maximum contrast between color of pavement and color of shoulder-surfacing aggregate, such as light-colored rock aggregate shoulders alongside hot-mix asphaltic concrete pavement. The more the color contrast, the less the shoulder is used for travel.

STRUCTURES AND HYDRAULICS SECTION

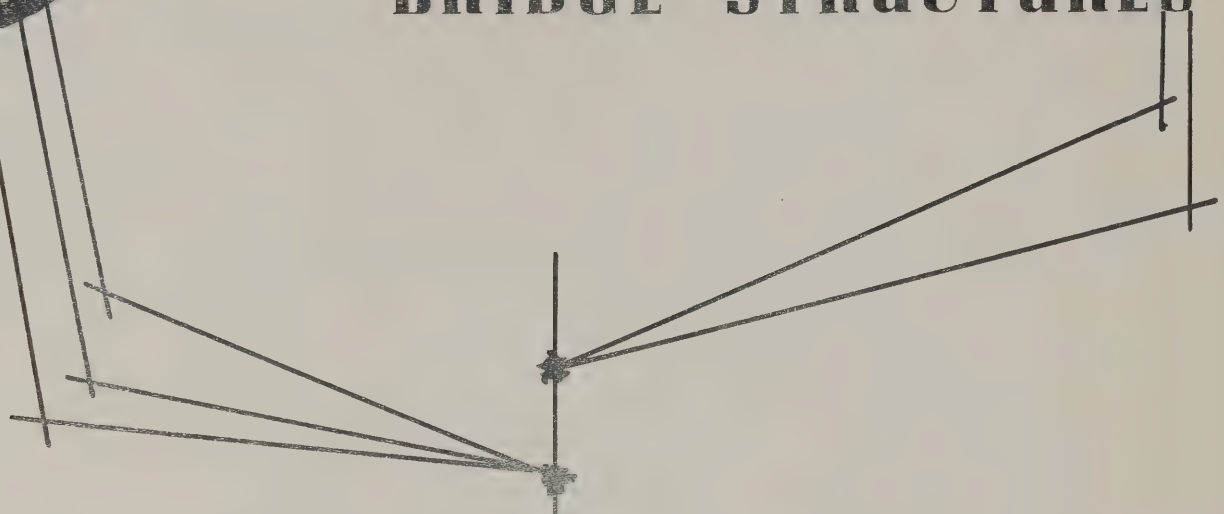
Morning: Room 2-B

Chairman: R.M. Holcomb, Professor of Structural Engineering
A & M College of Texas

9:00 Design of Welded Bridge Structures
Farland C. Bundy, Designing Engineer
Texas Highway Department
Austin, Texas

10:30 High Tensile Bolts to Replace Rivets
Wayne Henneberger, Senior Designing Engineer
Texas Highway Department
Austin, Texas

DESIGN OF WELDED BRIDGE STRUCTURES



Farland C. Bundy, Designing Engineer
Texas Highway Department
Austin, Texas

The welded bridge is making rapid strides of progress in our Highway Department. We are continually increasing span lengths, deepening girders, and, in general, making more efficient use of this versatile process for joining structural members. Texas, however, has not always been so forward in the use of welded bridges. About 25 years ago, when our Highway Department was expending its most ambitious efforts on concrete arches and simple trusses, the first large-scale use of welding for bridges began in France and Germany. Many of these all-welded highway and railway bridges built in Europe prior to World War II are still giving satisfactory service. The few that have failed served to indicate ways for improving the design of subsequent bridges. All of these failures can be attributed to one of three things: poor design, improper welding technique, or unsuitable base material. Much has been done to correct these early shortcomings and to educate the engineer to accept arc welding as a proven process.

About 1935 Canada and the United States began to build a few all-welded bridge structures. Connecticut, California, and Kansas were some of the states taking the lead in this construction. One of the factors retarding the growth of welding in some states was the reluctance of the Federal Government to grant Federal Aid on all-welded, built-up girders. This and other factors have contributed to the reluctance of American engineers to accept arc welding as one solution to their bridge problems. It was not until 1944 that the Texas Highway Department took some positive steps toward making the welded bridge a reality in our state. In the latter part of that year letters were sent to each District Engineer, requesting him to send a representative to a school to be held in Austin in January of 1945. This school was to be conducted by an expert secured from the Lincoln Arc Welding Company and was to acquaint the design personnel of the Department with the fundamentals of arc welding as applied to bridges. The school was a success and soon thereafter the Department began specifying welding for all diaphragms and bracing on I-beam bridges. In 1946 and 1947 some continuous I-beam bridges were

designed with welded field splices in the main members. These bridges were in selected residencies with private laboratories doing the welding inspection. By 1948, all continuous I-beam units carried the welded beam splice as a basis of pay with a riveted splice option. It soon became apparent that the contractors were favoring the welded splice and that the Highway Department was in dire need of more trained inspectors to handle this work. Thus, in 1948 Mr. P. V. Pennybacker of the Bridge Division began holding welding schools in the various districts to train men as welding inspectors. To supplement this program, several men were employed by the Bridge Division to do the welding inspection on highway structures throughout the state, when so requested by a District Engineer. The Austin office still has four such men available to do this work.

Our first all-welded plate girder was designed and put under contract in 1950. This was a three-span continuous unit with end spans of 100 feet and center span of 130 feet crossing the Sabine River between Smith and Upshur Counties. Welded field splices were called for at points along the girder about 30 feet in both directions from the interior piers. This bridge with some modifications served as a pattern for several later structures.

Early in the following year, a contract was let for an all-welded, grid-type structure over a railroad yard in El Paso. By grid we mean a system of transverse beams of equal importance to the longitudinal beams in transmitting load to the substructure. The conditions governing the design of this structure were very rigid, with the location of most of the substructure being fixed by the clearance lines of a series of closely spaced railroad tracks. This overpass had stringers and floor beams of equal depth welded rigidly together and supported by steel H-columns welded to either the stringers or the floor beams. Although this design was by no means ideal, it did give a workable solution to a difficult problem.

In the past four years we have completed about a dozen all-welded plate girder bridges and several others where welding was used extensively. Several recent developments, which we will discuss later, make the all-welded plate girder even more versatile than it has been in the past, and make it economically feasible for spans almost twice as long as our earlier thinking indicated. Our Department is truly on the threshold of tremendous advancement in welded bridge construction.

Before proceeding to a discussion of design methods, we should justify the use of welded connections rather than riveted or bolted ones. Where welding is used we may normally expect to accomplish the following results:

1) Reduce the size and weight of structural members and their connections. Full areas of tension members and of tension flanges of beams and girders are effective in carrying stress. Metal in a welded structure can be placed where it will function more efficiently. Splice plates are eliminated. These and other factors have combined to give a maximum weight saving of about 20 per cent in our built-up girders in bridges with spans from 100 feet to 150 feet.

2) Eliminate most if not all of the punching, drilling, reaming, and counter-sinking of holes, thereby reducing fabricating time and expense.

3) Give greater freedom to engineers and architects, making it easier for them to meet service requirements and, at the same time, design structures that are pleasing architecturally.

4) Produce smoother surfaces which generally mean less corrosion, lower painting and maintenance costs.

5) Reduce time for preparation of detail drawings, time for fabrication and time for erection because of lighter pieces and simpler details.

6) Pass on the weight saving in the superstructure to the design of the substructure. This can be quite important at a site when the safe bearing value of the soil is low.

One point should be clarified when

comparing welded and riveted plate girder units. In the riveted design of moderate span bridges fewer girders with a floor system give greatest economy; whereas in the welded design for the same span and same service conditions, economy comes from increasing the number of girders and eliminating the floor system. This makes for simpler, lighter pieces which are much easier to handle in the shop and much easier to transport and to erect.

A recent amendment to our specification, "Metal for Structures," and a forthcoming change in the American Welding Society Specifications has increased the possibilities for welded design in our bridge work. All of our welded highway bridges are now designed in accordance with the American Association of State Highway Officials in 1953 Standard Specifications for Bridges, which specifications in turn refer to the 1947 Specifications for the design of the welded features of the structure. This current amendment says in part that Structural Steel designation ASTM A 373 T shall be used in lieu of ASTM A 7 steel in all main load-carrying members which are made up by welding plates or shapes together or which are connected or spliced by welded connections. Briefly, this new A 373 steel differs from the A 7 steel in that it specifies the carbon, manganese, and silicon content for various thicknesses of plate in order to assure a fine-grained, weldable steel. It has been found that the fatigue strength and notch toughness of the weld as well as the weldability of the base metal are intimately related to the carbon and manganese content of the steel. The phosphorous and sulfur content and the tensile requirements are practically the same for both A 373 and A 7 steel.

The forthcoming revision of the A. W. S. 1947 Specifications will apply to plates up to four inches in thickness whereas the current specifications only go up to a two and one-half-inch thickness. This revision will not require mandatory furnace stress relieving for any thickness up to four inches and has only moderate preheat and low hydrogen electrode requirements. Whereas our designers have been limited to flange plate thicknesses of from one and one-half to two inches, they may now go up as high as four inches, with the proper precautions.

In discussing the design of welded bridge structures we shall place major emphasis on welded I-beam units and welded plate girder units with some remarks on miscellaneous types of structures. We shall further limit ourselves to the superstructure design of these structures. Although some very fine welded arches, trusses, and rigid frames have been built throughout the world, our Highway Department sees fit to emphasize the continuous girder bridge for its all-welded construction.

The continuous I-beam unit finds its greatest economy in spans ranging from 40 feet to 90 feet. When a preliminary layout study indicates that spans within this range are needed for a particular crossing, the continuous welded I-beam unit will most likely be the best solution. The layout should be made with full consideration of the service requirements as well as the desirable span ratios for a welded unit. The most efficient use of metal will generally result when the interior spans are from two-tenths to three-tenths longer than the end spans. When a tentative span arrangement is selected, the actual structural design may be initiated. In abridged form the steps of this design procedure are given below.

1) Based on roadway width and sidewalk, curb or median heights and widths, the beams are spaced to share more or less equally in carrying the slab dead load. The stringers should not be less than six nor more than ten feet apart. The slab design should be made for the girder spacing selected in order to determine the required thickness and weight of the slab. The dead load of the slab is distributed to the stringers and added to the estimated weight of structural steel carried by each stringer to give the total dead-load weight per linear foot of stringer.

2) The live load is made up of

standard H or HS trucks and equivalent lane loadings to give the effect of a truck train. The class of highway determines the type of loading to be used on the bridge. These loads are distributed to the stringers in accordance with the AASHO Specifications. Both lane loads and truck loads should be considered.

3) Any accepted method for computing shears, moments, reactions may be used when investigating the beams under the action of the dead and live load. A constant moment of inertia may generally be assumed for calculating these functions without introducing serious errors. Three of the more common methods are: a) Use of constant moment of inertia influence lines; b) Moment distribution procedure; and c) Three moment theorem analysis. The live load effect at any point should be considered separately for both truck and lane loading and the one giving the maximum should be used. The live-load stress should be increased by the appropriate impact percentage.

4) The dead load, live load, and impact effects should be combined to give values for plotting a maximum combined moment diagram and to give values of maximum shears at the supports. The shearing stress and web-crippling should be checked at the support to determine whether or not stiffeners will be required. The maximum moment diagram is used to determine cover plate lengths and cut-off points and to aid in locating splices. The splices are located near points of minimum moment to insure that the splice will never be very highly stressed and thus give a greater safety factor on the welded connection. The splice is made with a double-vee butt joint in both the web and flanges with smooth notches cut from the webs at their juncture with the flanges to insure better flange welds. Run-off pads or extension bars should be used to secure full development of the flanges.

5) The length of the beams for mill order should be limited to something under 60 feet wherever practicable, to avoid extra charges. Shop-fabricated lengths may be as great as 75 feet, but preferably should not be over 60 feet. Good judgment should be exercised in determining the number of shop and field splices and the number of pieces to be required.

6) If heavy cover plating is used or if different weight beams of the same series are spliced together in lieu of cover plating, a new analysis taking into account the changes in moment of inertia may be necessary to compute more accurate values for the bending moment. This is generally not necessary, since the addition of cover plates on most I-beam units will not change the bending moment values over 5 per cent from those found by assuming a constant moment of inertia. A check should be made on the live-load deflection of the spans to see that it does not exceed one-eight hundredth of the span length of any span. On infrequent occasions the live-load deflection will govern rather than the stress. At such times, deeper or heavier beams will have to be used to satisfy the deflection requirement.

7) The fixed and expansion conditions at bearings, along with the joint openings at the ends of the unit, should be decided on. Finger-type joints will be used where the maximum opening is two inches or greater and armor joints will be used where the maximum opening is smaller than this. Shoes should be welded to the bottom flange of the beam, preferably with down-hand fillets along the edges of the flanges. If cover plating prevents this, an overhead fillet across the flange on both sides of the shoe is permissible.

8) The final feature of importance is the selection and location of diaphragms or cross bracing. They should be placed at the ends of the unit, at all construction joints and at or near the interior reactions. They should be spaced a maximum of fifteen to twenty feet apart. A construction joint arrangement should be selected to meet the above requirements, yet use a minimum number of diaphragms. The length between construction joints (construction joint diaphragms) should be made a maximum of about 40 feet. The function of diaphragms should be kept in mind in their design. Any

diaphragms at reactions should be able to transmit the lateral forces in the slab or upper chord into the shoes. In addition, the end diaphragms as well as construction joint diaphragms should be able to support the discontinuous edge of a slab under live load. Construction joint diaphragms and x-bracing diaphragms serve to make the stringers act together in carrying live load. Also, these diaphragms give lateral support to the compression or bottom flange of the beams near interior supports. The bottom struts and cross bracing of all diaphragms should be designed as compression members, with limiting l/r ratios, to carry the calculated stress under the functions described above. The welded end connections should be designed for the calculated stress in the member, but never less than 75 per cent of the effective strength of the member. Fatigue conditions are generally not present to a critical degree in our highway bridges. During the life of the structure there will be very few repetitions of loading which would produce maximum stress in a welded connection either in the main members or in the secondary members. Reversal of stresses in the welded-beam splices are small in magnitude. The allowable unit stress in a butt weld is the same as prescribed for the base material except that for shear the normal allowable unit stress is 13,000 pounds per square inch. Butt welds are assumed to behave the same as the parent metal in all respects. Fillet welds, however, are designed for shear on the effective throat of the weld. The allowable unit-shearing stress is equal to 69 per cent of the allowable stress of the base material in tension (i.e., .69 x 18,000 = 12,420 lb./sq. in.). For example, the allowable stress per linear inch for a normal 5/16-inch fillet weld may be computed as follows: effective throat = .707 x 5/16 = .221 inch. Allowable stress = .221 inch x 12,420 = 2,750 pounds per linear inch of 5/16-inch weld. A sufficient length of weld should be provided to give the desired strength for the connection.

The plate girder units built to date might be termed overgrown I-beam units with only a few major differences in the design procedures for these two types of bridges. The girders are made up of thin web plates and constant-width flange plates which change in thickness to meet the bending requirements. The girders are spaced in much the same way as explained for I-beams to avoid a floor system and give greatest economy. Considerable study should be given to the selection of the most economical depth girder, taking into account such factors as grade lines, required waterway openings and freeboard or clearances in overpass structures, weight saving in structural steel and aesthetics of the structure. In constant depth girders the depth should be from 1/25 to 1/30 of the longest span length. Where longitudinal stiffeners are not used, the thickness of the web plate is required to be no less than 1/170 times the depth of the beam. The girder should be made as deep as possible for each 1/16-inch increment in web thickness (for example, a girder with 5/16-inch thick web should be made 52 to 53 inches deep for greater steel economy). The flange width should be made about 1/4 to 1/5 the depth of the girder, with a minimum of about twelve inches to provide torsional and lateral stiffness to the girders during erection. The wider flange also increases the safety of walking on the girders in the air. The moments, shears, and reactions should be found by considering the actual moment of inertia of the sections used. Two analyses will generally be necessary. Plate thicknesses may be changed to satisfy bending requirements at any point the designer feels the metal saved will offset the cost of a butt splice. Cover plating may also be used if felt to be cheaper than splicing in a heavier plate. In the past we have used cover plates to avoid having a flange plate thickness over one and one-half inches. However, with the use of the submerged arc mechanical welding process as well as with the low hydrogen electrode, we should be able to use heavier flange plates and avoid cover plating almost altogether.

Heretofore we have specified a double fillet weld on the web-to-flange connection and as such it has been limited in size by both the web and the flange-plate thicknesses. With the submerged arc process we have been getting a full penetration square butt weld in this joint. Use of the submerged arc process will be specified on future plans as an alternate to the fillet welds where a thick flange plate (say two inches or greater) is to be welded to a thin web.

A great deal can be done to avoid thick flange plates by deepening the girder over the interior supports. This causes a redistribution of the stresses along with a more efficient arrangement of metal to resist these stresses. The web may also be kept thin in the deepened section of girder by the addition of small longitudinal stiffeners.

Because of the thin web, vertical stiffeners are needed throughout the length of the girder to avoid web buckling. These are placed in accordance with the specifications depending on the amount of shearing stress in the girder web at the point in question. These stiffeners are generally plates welded in pairs or singly on alternate sides of the web. For outside or facia girders these stiffeners are all placed on the inside of the girder. They should be a minimum of four to six inches wide and a minimum of 5/16-inch thick and should be fitted tightly between the top and bottom flanges with the inside corners clipped to prevent fouling the girder fillet. They should be welded to the compression flange of the girder and to the web of the girder with intermittent fillet welds. Where a diaphragm frames into a stiffener, the stiffener should have additional weld to transmit the diaphragm stress into the girder web or flange. Bearing stiffeners are provided in pairs on opposite sides of the girder web at all bearings. They are milled to bear, top and bottom, made wide enough to extend near the edges of the girder flanges, and should provide an area sufficient to transmit the entire reaction into the shoe. This stiffener should be welded intermittently to the web and to the compression flange (to both flanges at end bearings).

The diaphragm functions and arrangement for plate girders are about the same as for I-beam units except here diaphragms may connect to stiffeners rather than directly to the girder webs. This provides greater tolerance in erecting the unit. In framing the diaphragms to the girders, no welds should be made across the tension flange of the girder. Also overhead welding should be avoided, especially on the bottom strut.

Another point of difference with the I-beam unit is the requirement of lateral bracing for the larger girder units. This bracing should be used where the spans are longer than about 125 feet or where the depth of the girder is greater than about six feet. It may be used on shorter spans if high water or wind conditions warrant it. The lateral bracing may be either a compression system, a tension system, or both. The laterals may be broken at girders and tied to the web near the bottom flange (or welded directly to the bottom flange in the compressive zone near interior supports). Shop splices may be made in the flange plates and in the web plates as needed to utilize standard mill lengths of plate. As in I-beam units, the field splices are made at the same point along the girder in both the flanges and the web (i. e., they are not staggered) and should be made near points of contraflexure in the girder. The ends of the girders are prepared for welding in much the same manner as the I-beams with the web being notched at its juncture with the flanges. If the ideal splice location causes shop lengths of girder which are too long to be shipped, then splices may be shifted to points of moderate bending stress and the splice made with high-tensile bolts. More will be said about this type of connection by the next speaker. It is difficult to place an absolute maximum on the fabricated length of girder section. We are at present considering the design of a girder which will have sections from 120 to 130 feet in length.

Welding has been used on several other types of structures by our

Department. Repairs and strengthening of existing trusses and beam spans have been used to extend the life and increase the inventory rating of many of our bridges. The arc-welding process has made salvage steel function more efficiently for us in several ways. Lugs have been welded to the top flanges of salvage beams to make the slab act compositely with the beams in carrying load. Salvage beams have been strengthened in various ways to increase their load-carrying capacity. Use has been made of the expanded beam idea on several jobs. In this construction, the web of the beam is cut along a serrated line, the halves of the beam are spread apart and the teeth are welded together. Some slides will be shown on this type of construction.

Composite action between the concrete slab and the steel girders is used by many highway departments on both simple and continuous units. Various types of lugs are used to insure this interaction. Composite construction can be used to good advantage in the positive moment regions of long girders to reduce live load deflections. Otherwise, the added costs of the lugs just about offset the steel saved by using composite action.

Welding is used on many of the minor features of bridge construction, such as shoes, railing, finger joints, armor joints, blast plates and others. In each instance where it has been used it has brought economy and simplicity.

Designers often disagree on the proper welded connection for a given condition. This is partly because they are hampered by working under specifications which were written for simple spans, and for riveted construction, and partly because of lack of precedent in welded construction. In considering the aesthetics of a structure, we will find even greater room for disagreement but we should nonetheless mention some of the architectural features of the welded bridge. The appearance of the girders and substructures should be given special consideration on all overpasses, expressway structure and large structures subject to public view. Welding provides for clean lines and light appearance of the superstructure. The accompanying railing, wing walls, and substructure should not be overly ornate or massive in keeping with the appearance with the girders. Drains should be placed to prevent their discharge from discoloring the outside girders where possible. No attempt should be made to hide the shoes or bearings. Much can be done with the railing and rail ends to add to the appearance of a bridge.

From all of the foregoing remarks we should not construe that welding is the cure for all bridge problems. We still feel that a great many of the larger structures such as trusses, steel arches, and some long-span girders can best be built by riveted construction. Welding loses much of its advantage when field splices have to be made at points where severe reversal of stress occurs under passage of live load. Welding involves more human error than riveting. Even with qualified welders and intelligent inspection, faulty welds will sometimes be made. Field welds should not be made in high wind, in rainy weather, or in extremely cold weather. Perhaps in the future a shop-welded and field-riveted girder would give us an economical structure with the greatest over-all safety factor.

The designer must weigh the worth of his structure as seen through the eyes of all interested parties. He must first consider the adequacy of the structure and its over-all economy. He must take the fabricator's view and see that it can be assembled rapidly and adequately in the shop. He must also consider the transporting of the various pieces from the shop to the field. He must further look at the structure from the erector's standpoint to see that the erection procedure is feasible and safe. He should also see the welder's view by making all welded connections accessible and by not calling for weld unless it is needed. The designer should finally view his structure as the ordinary citizen-taxpayer, whose use of a structure and opinions about it determine whether it meets the tests of service and beauty.

High Tensile Bolts TO REPLACE RIVETS

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Many of the bridges for which the Bridge Division of the Texas Highway Department is preparing plans are of the welded type. Welded fabrication has progressed to such a degree that both shop and field welding is the most economical type of fabrication for many of our structures. However, on some types of bridges, particularly long-span plate girder units and truss spans, we design for riveted splices and connections for both shop fabrication and field erection. On certain units which are presently in the design stage it is anticipated that we will require plate girders to be built up by welding the plates together in the shop but field splices and connections will be designed for riveting.

The purpose of this paper is to acquaint you with latest developments both within the Highway Department and on other construction projects in which high-tensile bolts are used in place of rivets. First, I would like to present a brief background on the development of high-tensile bolts and bolted connections followed by a description of several Texas Highway Department bridges presently under contract which are being field bolted; also, a brief account of other structural engineering work outside of the bridge field in which high-tensile bolts are being used. Quite naturally, the economics are of utmost interest in this new type of fastener. I have a few cost figures to present, but I'm in hopes that we can get some additional information from engineers here at the Short Course who have had experience in this work. Therefore, I would appreciate any questions which you have -- we will try to answer them--

and a general discussion at the end of this paper.

The use of bolts as a means of permanently connecting together the various members of a structural steel framework is not new. It has been an accepted practice with certain restrictions on unit working stresses and areas of application ever since structural members were rolled. The bolts ordinarily used in such connections are termed unfinished bolts. Common practice over the years has shown it to be economical to use unfinished bolts in loose-fit holes, but this type of bolt cannot be tightened sufficiently to eliminate the possibility of slipping under working shear loads. As a result, its use is confined to secondary connections where slip is unimportant. Elsewhere, rivets are specified in order to secure a better fit.

Another type of bolt frequently used in structural steel work, where clearances or limited quantities make rivets impractical or noneconomical, is the turned bolt. Specifications require that these bolts be installed in drilled or reamed holes with a close fit of 1/50 inch.

Still another type that has been used in limited quantities is the so-called rivet bolt with fluted or ribbed shanks; when properly installed with a driving fit, this type behaves much as do rivets.

As contrasted to the above, the use of high-strength bolts as a fastener in structural steel construction is quite new. Indeed, their use in such connections can almost be termed revolutionary, for they have been accepted by both structural and bridge engineers as a fastener suitable not only for static loads, but also for all ranges of dynamic loads. This idea, however, and its effects, did not emerge at one time, but only through a process of development and contribution by many engineers. Possibly the germ of this idea lay in the practice of certain fabricators a good many years ago of using high-strength bolts as fitting-up bolts. This provided sufficient clamping power to pull up heavy plates preparatory to riveting. The principle of this connection is also new. Loose-fit holes are retained for economy in assembling, the bolts being installed in holes 1/16 inch larger than their nominal shank diameter. The nuts are driven up with power wrenches producing very high tension in the bolts and providing more clamping force than is possible with rivets. This tension is depended upon to resist shear load through friction at the joint's faying surfaces and at working loads slip is not possible in view of the high friction produced between the joined members. Under normal conditions the connected material does not come into bearing on the bolts. The bolts are made of carbon steel heat-treated for strength, and a hardened washer is placed under both the head and nut to permit the development of the high torque required without scoring the connected material.

With this as an introduction to the subject of bolted joints I would like now to give you a brief bibliography on the research which has gone forward in this subject. Many people feel that for a long time the construction industry has needed a faster and better way to fasten connections -- some believe that the high-tensile bolt is the answer. Practices in the design of riveted and bolted joints have been developed largely from experience and have not always been supported by conclusive experimental data. With this fact in mind, twelve sponsoring organizations instituted the Research Council on Riveted and Bolted Structural Joints in January, 1947. The specific object of the Council is to determine the strength and action of joints, and to promote safer and more economical design. Important among the projects of the Research Council is the study of high-strength bolts in structural joints. Our work in this field today is based on results of investigations and recommendations of this Research Council. It is expected that the Council's work will result in the promulgation of more economical and more efficient practices. There were laboratory tests on high-strength bolts prior to the formation of the Research Council. The results of this work and of various projects under the Research Council's activities indicate that "the fatigue strength of high-strength bolts, appreciably smaller than the holes in the plates, was as great as that of well-driven rivets if the nuts were screwed up to give a high tension in the bolts." In a paper published in May, 1954, summarizing the results of a number of static and fatigue tests conducted at the University of Illinois on structural joints using high-tensile bolts test data indicate that these joints are generally superior to similar riveted joints whether subjected to static or fatigue-type loadings. The fatigue strength of high-tensile bolts is probably the most important question considered in tests. Results indicate that joints withstand in many instances 2, 000, 000 to 3, 000, 000 cycles of load application without failure. Even then the fatigue strength for failure is $\pm$20, 000 p. s. i. for the higher bolt tensions. Contrasted to these results similar tests made on riveted joints indicate that when subjected to a stress of $\pm$ 18, 000 p. s. i. joints failed between 578, 000 cycles and 713, 000 cycles. This is equivalent to a fatigue strength at 2, 000, 000 cycles of approximately $\pm$16, 000 p. s. i. This indicates that the bolted joints are about 25 per cent stronger in fatigue than similar riveted joints. In no case during these tests was there a fatigue failure of the high-tensile bolts in the joints providing there was no slippage of the joints during the application of load. It is of interest to note that in one series of tests, joints in which bolts were tightened to 320 foot pounds as recommended by the Research Council and tested at zero to 21, 500 p. s. i. range of stress no failure occurred even after 12, 275, 000 cycles of loading. Static tension tests of various types of joints indicate that joints with hot-driven rivets and those with bolts resist the load by friction between the plates until the friction is overcome. The fasteners come into bearing. Investigations by the Research Council indicate that rivets, both hot- and cold-driven, do not completely fill the holes in structural members, but provide a certain amount of clamping force between the rivet heads so that the load is transferred from one member to the other by friction. The clamping force produced by hot-driven rivets results principally from the cooling and consequent shrinkage of the rivets, and varies with the grip, becoming greater as the grip of the rivet increases. Greater clamping force can be developed by the use of high-strength bolts.

Usually structural plates fastened with rivets will fail on a net section through the first line of rivet holes but it is not unusual for the plates of a bolted joint to fail through the gross section on a line near the edge of the washers. This indicates that the clamping action of the bolts has reduced or eliminated the high concentration of stress around the holes.

For additional test results I refer to A. S. C. E. Proceedings published

as Separates Number 440, 441, 469, and 470. Some of the test results which I have given you above were taken from these publications.

In order to answer one of the most important questions concerning high-tensile bolts, the Association of American Railroads replaced rivets with high-tensile bolts in bridge members on several railroads to determine if the bolts would retain their clamping force under dynamic load conditions. Where it had been necessary to replace rivets in many instances yearly, high-strength bolts after six years retain their full clamping force without maintenance. Rivets were replaced by bolts in several floor-beam hanger connections of riveted trusses where the temperature falls to minus 40°F. These bolts after two winters have proven satisfactory.

The reliability of the structural joints fastened by high-strength bolts has been demonstrated by the laboratory and by field test. All designs prepared by the Bridge Division of splices and connections of bridge members are based on the use of rivets or welding. At the contractor's option high-strength bolts of the same nominal diameter as the rivet may be substituted, one high-strength bolt for one rivet. These bolts must conform to ASTM Specification A-325. As described above, these bolts are regular semifinished hexagon head; the nuts, heavy hexagon semifinished; and the washers, heavy, plain (carburized). The bolt heads are marked to denote high strength; the washers are hardened to prevent the bolt from cutting into the softer structural steel; and the nuts are made reversible. Beveled washers are made square for easy recognition.

Whenever size and length of bolt permits, full-size bolt-tension tests should be made and in this case the proof load shall be determined. Whenever full-size tests cannot be made, the yield strength instead of the proof load shall be determined. There is a test procedure outlined in the ASTM to determine the proof load. The length of the bolt after being loaded to the required proof load must not exceed that before loading by more than 0.0005 inch. It is intended that no permanent elongation is desired and the limit of 0.0005 inch is used to include measuring errors. Proof load may be determined on a full-size bolt using an autographic load elongation method. When this method is used, the minimum value at 0.2 per cent offset of the six-thread gage length shall be 93,000 p.s.i. for sizes one-fourth to one-half inch, inclusive; 88,000 p.s.i. for sizes nine-sixteenth to three-fourths inch, inclusive; 81,000 p.s.i. for sizes seven-eighths inch and one inch; and 74,000 p.s.i. for sizes one and one-eighth inch to one and one-half inch, inclusive.

Yield Strength (Turned Specimen): When determination of yield strength is required (when proof load cannot be determined on the full-size bolt) a standard 0.505-inch test bar shall be turned and tested using either an extensometer or autographic stress-strain method. The yield strength by the extensometer method shall be the stress at which a permanent set in the gage length of 0.2 per cent occurs. The yield strength by the autographic method shall be the stress at 0.2 per cent offset.

The ultimate load of high-strength bolts shall conform to the following:

125,000 p.s.i. for sizes one-half inch and under.

120,000 p.s.i. for sizes nine-sixteenth inch to one inch inclusive.

105,000 p.s.i. for sizes one and one-eighth inch to one and one-half inch inclusive.

Specifications require that faying surfaces when bolted together shall be free of paint, lacquer, dirt, oil, loose scale, burrs, pits, and other defects that would prevent solid seating of the parts or would interfere with the development of friction between the parts. The high-tensile bolts are usually tightened in the field by a manual torque wrench or by a power wrench. The power wrench or air impact wrench produces fast, accurate, and economical work but must be properly calibrated to produce the correct tension in the bolt. The recommended bolt tension for calibrating wrenches for three-fourths-inch bolts is 29,000 pounds, for seven-

eighths-inch bolts, 37,000 pounds. This is approximately 15 per cent in excess of the required minimum bolt tension. The required minimum bolt tension, which is 90 per cent of the minimum proof load (or elastic limit), is 25,600 pounds for three-fourths-inch bolts and 32,400 pounds for seven-eighths-inch bolts. The approximate equivalent torque for these required minimum bolt tensions is 320 foot-pounds for three-fourths inch bolts and 470 foot-pounds for seven-eighths inch bolts. Devices have been built for calibrating air impact wrenches. They are so constructed that when a bolt is inserted and tightened, two plates are pulled together, and the clamping force is measured by electric resistance strain gages or by hydraulic pressure and converted to bolt-tension readings. Since the specifications require that the bolts be tightened to at least 90 per cent of the elastic proof load and the wrenches vary by, say 15 per cent, the wrenches are set somewhat higher than the exact tension required. To regulate the power of the wrench, a pressure regulator is put in the feed line and set to the pressure which gives the required tension on the bolt as determined by the calibration. The impact wrench exerts all of the force of which it is capable, and then stops. The impact wrench must be checked several times during the day.

Inspection of bolts must be made after the tightening operation. Usually about 5 to 10 per cent are checked by a hand-operated torque wrench. The bolt is loosened and retightened so that when retorqued to its original position, the bolt has the torque required by the specifications. Of course care must be taken to see that the manual torque wrench is properly calibrated. The torque is measured while the nut is in motion. If the retightening discloses a high frequency of lower than specified torque in initial tightening of bolts, then the inspector should require retightening of all bolts in affected area. It is advisable for the inspectors to work close behind the bolt-tightening crews so that both can work together.

On a recent Atomic Energy Commission project in Ohio, three process buildings requiring 79,000 tons of structural steel were erected. Three companies erected the steel; one elected to use both high-tension bolts and rivets while the other two companies used high-strength bolts and machine bolts. It is estimated that a total of one and one-half million high-strength bolts were used in all. The average bolt-tightening crew of two men handled 350 to 450 bolts per shift while a riveting crew of four men averaged 175 rivets per shift and usually had about 15 per cent cut-outs. Inspection showed that less than one-half of 1 per cent of the bolts are off more than ±10 per cent of the set torque values. According to the author who described this project in the August, 1954, issue of Civil Engineering, the use of high-strength bolts speeded up the project. Riveters are usually in short supply, but bolting crews can be trained in a very short time. This new technique of bolting joints made possible the use of considerably more unskilled help than has ever been feasible before in steel erection.

During the past year high-strength bolts have been substituted for rivets in an increasing number of commercial buildings. The 23-story Mile High Center Building erected in Denver is one of the nation's tallest buildings using bolted construction. Erection of the steel work was completed in 82 working days and high-tensile bolts were used to fasten the structural joints throughout.

In Portland, Oregon, a fourteen story University of Oregon General Hospital was erected using high-strength bolts. This was one of the first steel frames in which the designers actually took advantage of the greater efficiency of these bolts over rivets. The magazine, Building Engineer, in a description of this project gave the following cost data: "The total installed cost per bolt came to 21 cents less than the cost of rivets, although the initial cost of each bolt is considerably greater — 30 cents vs. 7 cents. All high-strength bolts were three-fourths inch. The use of high-strength bolts saved 80 tons of

steel by developing up to 47 per cent of full continuity, thus adding useful rigidity to the structure and cutting the total weight by 7 per cent. Each two-story tier was assembled, lined up, and the bolts tightened in only two days. It was reported that riveting or welding this portion would have taken a week.

The Texas Highway Department has several large bridges under contract on which the contractors have elected to use high-strength bolts in place of field rivets. Two of these projects, the bridge across the west fork of the Trinity River on Loop 12 in Dallas County and the bridge across the Red River on State Highway 37 in Red River County, have just been bolted in the past few days. Erection of the main plate-girder steel across the Trinity River on State Highway 73 in Chambers County has not begun. This will be the largest plate girder erected on the Texas Highway Department System with high-strength bolts.

The 340-foot continuous-deck plate-girder bridge across the west fork of the Trinity River has span lengths of 100'-140'-100'. It is a two-girder system with floor beams and stringers. There are four field splices on each of the two girders. Approximately 5,400 three-fourths inch high-strength bolts will be used on this bridge. In a discussion with an erection foreman on the job during the first day of bolting operations, I was advised that they anticipated tightening 500 bolts or more per day with a two-man crew. It was necessary for the two-man crew to place some of the bolts as well as tighten all bolts in the splices and connections. According to my timing, it took five to eight seconds to tighten a bolt once the wrench was placed on the nut. Splices were made with pins and high-strength bolts, effecting a great saving in not having to use different erection bolts. Later the splices were to be filled out with additional high-strength bolts. Another saving was in the use of skilled iron workers at $2 per hour in place of riveters at $2.87 per hour. It was anticipated that even with the increase in cost of high-tensile bolts over rivets, $1,100 would be saved on erection of the steel. Possibly we can learn more from the engineers here who have been working on this project.

The 660-foot continuous deck plate girder across the Red River has span lengths of 150'-180'-180'-150'. It is a two-girder system with floor beams and stringers. There are six field splices on each girder. Approximately 8,600 seven-eighths inch high-strength bolts will be used on this bridge. The contractor on this job had all bolts placed ahead of the bolt-tightening crew, thereby speeding up the number of bolts tightened per day. I understand that approximately 1,800 bolts were tightened by a three-man crew the first day of bolting operations on this bridge. Here again it took only a few seconds to tighten each bolt. Apparently there were no serious difficulties experienced in the use of high-strength bolts on either of these projects. As is customary on plate-girder spans, all shop and field splices were subpunched or subdrilled and reamed full size while the connecting parts were assembled. It was necessary for the shop to omit paint on sections to be bolted together. Before bolting operations in the field, it was necessary to wire brush the surfaces to remove loose scale and rust.

It is my opinion that high-strength bolts will become fairly universal in the next few years. With continued research, improvements in materials, and more realistic designs for high-strength bolts, more people will understand the value of this type of fastener. An impact wrench which can be set to a predetermined shutoff to give the required bolt tension is being developed. This wrench will greatly benefit erectors by speeding up work and reducing the amount of checking required. A reduction in price of bolts through more production will make high-strength bolting more economical. There are other advantages and some disadvantages in the use of high-strength bolts. However, only after more use can the Texas Highway Department actually evaluate the benefits to be derived and the economy in use of this type of fastener on highway bridges.

PAVEMENTS SECTION

Morning: Ballroom

Chairman: F. M. Davis, District Engineer
Texas Highway Department
San Antonio, Texas

9:00 Jointing of Concrete Pavements
A. L. Chollar, Supervising Designing Engineer
Texas Highway Department
Austin, Texas

10:00 Resurfacing with Concrete
George M. Garrett, Special Highway Engineer
Portland Cement Association
Fort Worth, Texas

11:00 New Developments in Constructing Concrete Pavements
Ted Edwards, Paving Engineer
Portland Cement Association
Austin, Texas

Afternoon: Ballroom

Chairman: O. A. Philippi, Assistant Construction Engineer
Texas Highway Department
Austin, Texas

1:30 New Uses for HVMS Emulsions
J. C. Roberts, District Engineer
Texas Highway Department
Abilene, Texas

3:00 Design of Hot Mix Pavements
J. R. Harris, Supervising Field Engineer
Texas Highway Department
Austin, Texas

Hugh A. Wallace, Division Engineer
Asphalt Institute
Dallas, Texas

Jointing of CONCRETE PAVEMENT

A.L. Chollar, Supervising Designing Engineer
Texas Highway Department
Austin, Texas

One thing about joints in concrete pavement is clear. We have not found the Utopia. It also is clear that a concrete slab will either be constructed with joints or cracks will soon form in the slab.

As might be expected, more success has been achieved with some joints than others. Perhaps the most satisfactory joint is the longitudinal center joint. The first concrete pavements in Texas were constructed without a longitudinal center joint. These pavements developed a longitudinal crack which usually was found in the middle one-third of the pavement. To control this crack, a center joint with tie bars was designed. This design provides for a crack formed by one of several optional methods with tie bars to hold the crack together. The design permits the two halves of the pavement to act as a hinge. It also reduces warping stresses that would be objectionable without such a joint.

An analysis of the tie bars as used in Texas (one-half inch round bars on 30-inch centers) indicates the joint is not too rigid. Using usual analytical methods, we find a ten-inch slab will stress the tie bars to approximately 28,000 pounds per square inch. With a stress of this magnitude the tie bars probably elongate slightly and permit the hinge to work.

A closer spacing of tie bars has been tried and from observation of the results it was concluded that the joint was too rigid. If the joint is too rigid longitudinal cracks tend to form outside of the tie bars. With a greater spacing of tie bars the crack follows the center line of the pavement reasonably well. I believe the present Texas design for longitudinal center joints is satisfactory and that most of the failures can be traced to causes other than design of the joint.

Specifications require concrete pavement to be placed when the temperature is at least 35°F and rising. If we assume the temperature at the time of hardening is 50°F and that later the temperature of the slab increases to 130°F, we can calculate the compressive stresses that would result if the slab cannot expand. Stress equals 80 times the coefficient of expansion times the modulus of elasticity. The calculated stress for gravel concrete amounts to 1,760 pounds per square inch. This value probably is too high for the following reasons. Much of the pavement would be cast at a temperature higher than 50°F. Concrete shrinks upon hardening. If the slab length is not too long, a slight space is available after hardening for some expansion. The temperature increases slowly from winter to summer and plastic flow probably relieves stresses to some extent. Moisture in the slab is at a

maximum when the concrete hardens and a minimum during periods of maximum temperature. Actual observations indicate a stress of about 1,000 pounds per square inch under conditions where the pavement is not allowed to expand.

It appears that expansion joints may be omitted in concrete pavements constructed of sound aggregates (nonexpansive aggregates) if the following requirements are met:

1. Relatively short slabs are used.
2. Joints are not infiltrated.
3. Compressive strength of concrete is at least 3,500 pounds per square inch.

Short slabs permit the concrete to shrink during hardening without forming shrinkage cracks. If joints and cracks are infiltrated with incompressible material, the effect is the same as increasing the slab length without increasing the space available for the slab, and results in higher compressive stresses. A concrete whose compressive strength is 3,500 pounds per square inch allows a generous safety factor and permits unlimited repetitions of stress without fatigue.

A complete discussion of contraction joints involves the design of concrete pavement and this subject cannot be presented in the time available. We can, however, list some of the variables that are involved in the design of concrete pavements. They are wheel load, temperature and resulting warping, compressive and tensile stresses, modulus of elasticity, Poisson's ratio, type of subgrade, impact, fatigue of concrete, radius of area of load contact, coefficient of friction between subgrade and pavement, strength and quality of concrete, moisture content of concrete, and reinforcing steel. The interrelationship of these variables presents a very complicated problem and should be carefully studied before designing contraction joints.

A list of the desirable qualities of contraction joints may be helpful in selecting their design and spacing. It is believed that contraction joints should:

1. Open and close without stressing the pavement.
2. Provide load transfer at all times.
3. Be free from the harmful effects of corrosion.
4. Prevent infiltration.
5. Be economical and easily installed.

If warping stresses are to be relieved contraction joints or warping joints must be provided at relative short intervals. It also would be desirable to space contraction joints at such intervals that slab movements are small. If the slab movement is very small the opening cannot be easily infiltrated with water and the slabs can be made to transfer load by interlocking the slab ends.

A study of restrained temperature warping stresses for nine- and ten-inch slabs was made by Mr. H. G. Bossey when he was Assistant Engineer of Road Design. This study indicated that restrained temperature warping stresses build up gradually as the slab length increases from five to forty feet, then they remain constant. A five-foot slab has very small or no stresses. The maximum stresses are reached when the slab is 20 to 40 feet long, depending on the subgrade. Maximum stresses are reached on the best subgrades when the slab length is 20 to 22 feet. Maximum stresses are reached on the poor subgrades when the slab length is 25 to 40 feet. A ten-inch slab on a subgrade whose "k" value is 50 reaches a state of maximum stress when the slab length is approximately 40 feet.

Under favorable conditions the magnitude of the warping stress may be in the order of one-half the stress produced by present wheel loads. If by design we can reduce this stress, the effort will be worth while. From a practical standpoint a slab length of twelve feet appears to be about the minimum which should be considered. The usual pavement is 24 feet wide and has a longitudinal center joint. If a twelve-foot slab length is used, then the effective width of the slab equals the length. A good argument could be made for a slab of this length. However, the closer we space the contraction joints the more we increase the cost. The spacing of axles on heavily loaded vehicles is such that two heavily loaded axles could not rest on a fifteen-foot slab at the same time. Shortening a slab from 40 feet to 15 feet would reduce the maximum restrained

temperature warping stresses of a ten-inch-thick pavement 60 per cent. A fifteen-foot slab would not require reinforcing steel to help the slab adjust itself on the subgrade during periods of temperature reduction. Using usual methods for analyzing and calculating the stress caused by temperature reduction, we find that only twelve pounds per square inch stress is required to adjust a slab fifteen feet long during temperature reductions.

In the early thirties, Mr. E. C. Woodward made a crack and joint survey on pavements constructed in the Waco area. The surveys were started as soon as possible after a job was finished and a record of cracks was kept. The pavement design provided for six-inch thickened edge slabs from 40 to 120 feet long. I believe the results of that survey were never published; however, during the progress of the survey it appeared that cracks occurred at a distance of twelve to fifteen feet from the slab ends very soon after the jobs were completed.

On some recent projects in Texas where the pavement design provided for contraction joints at fifteen-foot centers, alternate joints were omitted for as much as 150 feet to find out what a 30-foot slab length would do. In those cases I have observed a transverse crack has formed in the 30-foot slab within a year or two after the completion of the work.

In June 1951 the first fifteen-foot corrugated contraction joint pavement in Texas was opened to traffic. Since that time a number of jobs using this design have been completed. I have not been able to examine these jobs as closely as I would like to, but as yet, I have not seen a transverse crack in a fifteen-foot slab.

The fifteen-foot corrugated contraction joint design does not provide for sealed joints. Several attempts have been made to find a way to seal the small crack but so far no entirely satisfactory method has been found. Bids will be received in September for a job in the Fort Worth District that provides for constructing a groove in the plastic concrete and filling the groove with a seal.

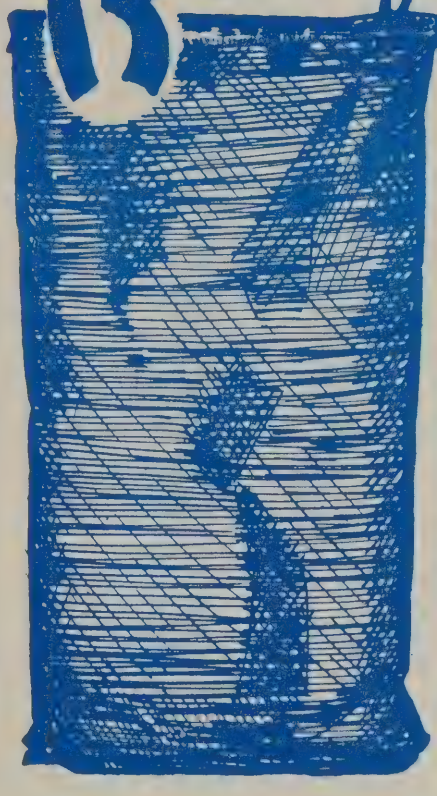
It has not been proved that a seal is necessary where the corrugated contraction joint design is used. The opening is so small and circuitous that water will not readily travel through the joint. Furthermore, until the edges and shoulders are sealed, the joints -- if sealed -- would not stop water from getting under the slab. A seal would prevent infiltration of dirt and foreign matter that could cause spalling and possibly blowups.

On recent projects where fifteen-foot contraction joints were used the joint has been formed with a metal form vibrated into and left in the pavement. Corrugations in the form provide alternate tongues and grooves that interlock to provide load transfer and prevent faulting. The metal form insures that no tensile stress will be transferred across the joint during the hardening of the concrete and provides positive crack control.

Another type of contraction joint has been used. This joint is formed by grooving the concrete to approximately one-third its depth while the concrete is plastic. The cost of this type joint is less than the cost of the joint formed with a metal form, but it has several objectionable features. There is a possibility that the plastic concrete will flow together after grooving so that a true weakened plane is not formed and no crack forms or if it does form it may not follow the groove. Load transfer depends on aggregate interlock which, after a few cycles of expansion and contraction, may break down. The crack does not form until the pavement is stressed to about one-half its total strength away from the groove, thus permitting tensile stresses to be transferred across the joint during hardening.

It is still too early to form definite conclusions on the fifteen-foot contraction design for concrete pavement. Unfortunately it takes ten to fifteen years to prove a design. By that time the changed conditions make an evaluation difficult. In conclusion, I recommend that the engineers in the field report their observations of the behavior of the design. If this is done it will contribute a great deal to the improvement of design.

Resurfacing With Concrete



There are many miles of old pavements in Texas twenty to thirty years old that have value and usefulness and can be continued in use for many years if they are properly resurfaced. There are three principal reasons for resurfacing; (1) to correct roughness, (2) to increase load carrying capacity, and (3) to correct grade line or widen, or a combination of all three reasons for doing the project.

Resurfacing old pavements with portland cement concrete is now recognized and accepted by many states, cities, counties, and the U. S. Bureau of Public Roads as a highly

George M. Garrett
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Fort Worth, Texas

satisfactory and economical means of lengthening the useful life of old pavements. The amount of patching that can be done economically is normally dependent on local conditions; but as a general rule when the area to patch amounts to two per cent to three per cent of the total area, then resurfacing is the logical answer. Also, if a thin pavement has been giving satisfactory service but there is an increasing traffic volume and increasing traffic load, serious consideration should be given to strengthening the thin pavement by resurfacing with portland cement concrete.

There are many miles of old 9-6-9 inch or 8-6-8 inch concrete pavements that have been giving satisfactory service for the last fifteen to twenty five years. But who could forsee the tremendous weight of commodity trucks that would be placed on our highways during World War II and in the period of industrial expansion after the war. These heavy loads have caused miles of these underdesigned pavements to begin to show distress. Now is the time to reinforce and increase the life of these old pavements by resurfacing with concrete paving. The postponement of resurfacing may result in having to construct a thicker resurfacing than if it is done before the pavement shows too much distress.

Some of our Texas highway engineers recognized this years ago and you can find projects of resurfacing with concrete pavement that are eighteen to twenty years of age on some of the state highways. In 1936 a four-inch concrete resurfacing project was constructed on U. S. 69 in Center, Texas. This concrete resurfacing is still in good condition. In 1945 there was constructed a concrete resurfacing project on U. S. 82 west of Paris, Texas. This was also a widening project. This pavement is in excellent condition at this time.

In this paper, we will discuss briefly the use of four-inch thick or thicker concrete as a resurfacing for an old pavement. Then if we have time, we will discuss a recent project in which one-half-inch to two-inch concrete resurfacing was used.

RESURFACING WITHOUT WIDENING: The old pavement should be cleaned of bituminous materials. In general only those patches that might shift or decay after the resurfacing is placed should be removed.

If the old pavement has adequate width, the resurfacing needs to be only the width of the old pavement. However, most of the old pavements that need resurfacing also need to be widened and this should be done integrally with the resurfacing. This design avoids a thin resurfacing thickness on the edge. Experience and observation show that it is not necessary to place special reinforcing in the resurfacing slab over the edge of the old pavement.

If the resurfacing is to be placed over an old flexible type of pavement, the "k" value of the old flexible pavement should be determined (see pages 9 through 12 of Concrete Pavement Design for Roads and Streets by the Portland Cement Association). Then proceed to design the concrete resurfacing the same as you would a regular concrete slab. (See pages 18 through 28 and pages 34 through 40 of the same design book.) If the computed thickness is less than four inches, it is recommended that a minimum of four inches be used. The construction procedure of concrete pavement on flexible surfacing is the same as for regular concrete pavement.

It has been the practice to design the resurfacing course on concrete pavement on the theory that the strength of the two slabs is equal to a single slab that has the thickness of the square root of the sum of the squares of the individual slabs. For example, if you have a six-inch slab with a five-

inch overlay or resurfacing, it is theoretically equal to

$$\sqrt{6^2 + 5^2} = \sqrt{61} = 7.78"$$

or a slab of 7.78 inch thickness. Or if you desire to arrive at the recommended thickness of a concrete resurface course the equation is expressed as

$$d_r = \sqrt{d_n^2 - Cd_e^2} \text{ (Formula A)}$$

d_n = the thickness required of a single slab designed for prevailing wheel loads and placed directly on the subgrade; the working stress for the concrete to be equal to that of the resurfacing.

d_r = the thickness of the resurfacing.

d_e = the thickness of the existing slab.

C = coefficient of condition of the existing slab after all the repairs have been made.

C = 1.0 if the existing slab is in good condition and all necessary repairs made.

C = 0.75 if the old pavement has inadequate joint design or has only initial joint or corner cracks.

C = 0.35 if old pavement is crushed or badly cracked.

The value of "C" is entirely up to the judgment of the engineer. It is also up to the engineer to decide the amount of expenditures that should be made in repairs to increase the value of "C" -- this repair expenditure to be weighed against the cost of increasing the thickness of the resurfacing course. The above formula should be used in all cases where there is a separating course of bituminous material and no widening is planned and where traffic will be using the edge of the resurface course.

The placing of a granular or bituminous course between the old pavement and resurfacing should not be used. The cleaner the old pavement, the better. However, in most cases it would not be economical to remove old bituminous treatments. Neither is it necessary to make special or extra effort to secure bond between the old and the new concrete slabs. However, the old pavement should be cleaned and dampened prior to placing the resurfacing course. It is advisable to always attempt to obtain the best bond possible. Investigations have revealed that in many cases the bond between the old pavement and the new concrete resurface course was sufficient to cause them to act as one slab of full thickness.

RESURFACING AND WIDENING: If the resurfacing is to be placed on a wide pavement or widening is to be done integrally with the resurfacing, a different formula should be used:

$$d_r = \sqrt{d_n^{1.87} - Cd_e^2} \text{ (Formula B)}$$

Assuming the d_n should be nine inches and the d_e is six inches and the present pavement is twenty feet wide and it is desired to widen the pavement to twenty-four feet and also strengthen by resurfacing with concrete pavement, the above formula should be used. Assume C = 1.0

$$d_r = \sqrt{9^{1.87} - 1.0 \times 6^2} = \sqrt{61 - 36}$$

$$= \sqrt{25} = 5"$$

Using Formula A

$$d_r = \sqrt{81 - 36} = \sqrt{45} = 6.7"$$

or 6 3/4"

This is a difference of 1 3/4 inches.

The reason for assuming C = 1.0 is because it is so simple to make all major repairs by removing old broken and crushed section full width and replacing with full-depth concrete pavement. This would put the old pavement in top condition. It is recommended that load transfer dowels be provided between the old pavement and the new full depth pavement.

CROWN: If it is desired to reduce the crown in the pavement, the minimum or calculated thickness should be used at the center of the resurfacing. At the beginning and the end of the new resurfacing, it will probably be necessary to remove some sections of the old pavement in order to match the grade and crown lines.

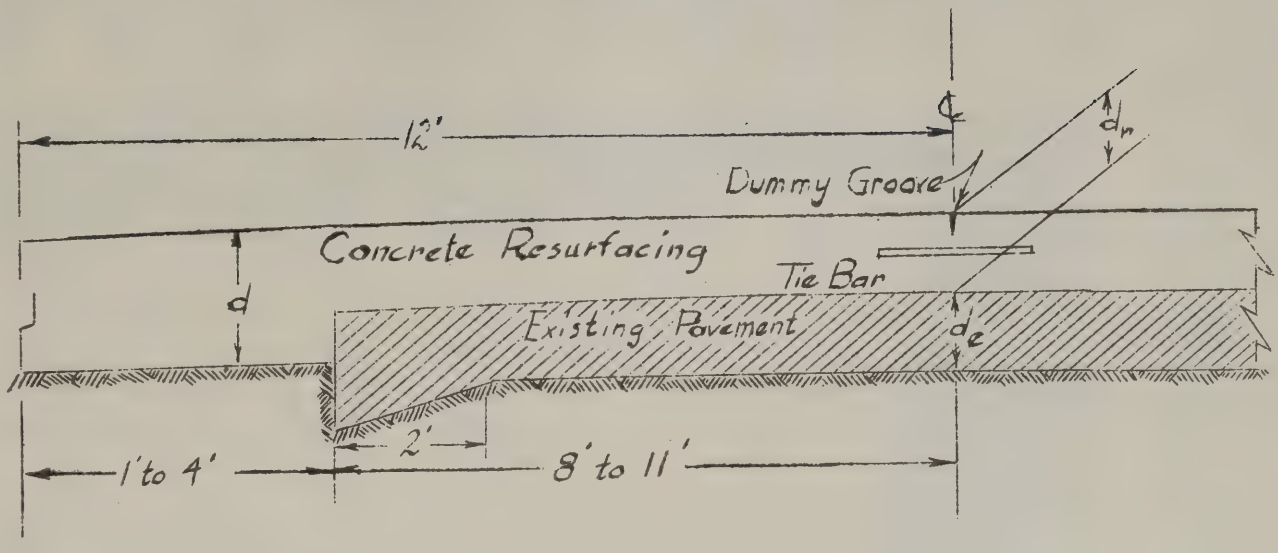
JOINTING: No fixed standard for jointing can be prescribed. The design should include both longitudinal and transverse joints. It is found that cracks will occur in the resurfacing above joint or open cracks in the old pavement. Therefore, grooved or sawed joints should be provided in the concrete resurfacing above all old joints or open cracks that are in the old pavement. If transverse joints are too far apart in the old pavement, some provision should be made to provide transverse joints at approximately twelve-foot or fifteen-foot intervals and not over eighteen-foot spacing. Longitudinal center joints should be provided above the longitudinal center joint in the old pavement. Joints in the resurfacing need not be of the same type as the joint in the old pavement. It is not necessary to provide expansion joints every 60 feet or every 100 feet in the resurfacing even though the expansion joints are of this spacing in the old pavement. Expansion joints in the concrete resurfacing can usually be placed at long intervals or be omitted entirely except at intersections or structures. Contraction joints can be placed directly over or within six inches of the old expansion joints, contraction joints, or wide-open transverse cracks that extend across the pavement. When expansion joints are omitted in the resurfacing and contraction joints are provided at fifteen-foot intervals, adequate load transfer can be expected by the interlocking of the normal size coarse aggregate when a dummy or sawed groove is provided, or load transfer can be provided by the use of the corrugated contraction joint.

Longitudinal hinged joints of surface dummy grooves with tie bars should be provided to divide the resurfacing slab into lanes of not over thirteen feet in width. Should only one lane be constructed at a time, keyed joints could be provided at the edges to help provide load transfer to the adjacent slab.

REINFORCING: Plain concrete has proven satisfactory for resurfacing where there has been no abnormal breaking or cracking. Where abnormal cracking has developed, the use of reinforcing is desirable. When reinforcing is used, it should be of small diameter to avoid the risk of the concrete cracking over the bars. Reinforcing should not extend across any transverse joints.

CONSTRUCTION METHODS: In general, the placing of concrete for resurfacing is the same as placing a new slab. However, there are certain items which require special attention.

Water: Since the resurfacing is to go over old pavement, there will be no absorption of water from the mix. For this reason, the mix should be designed with less water. Where widening is to be made integrally with the resurfacing, the subgrade for the widening should be thoroughly wetted to avoid undue absorption of water from the mix. On some projects, the placing of building or roofing paper on the subgrade is done to assure the same absorption factor as might be expected on the old pavement.

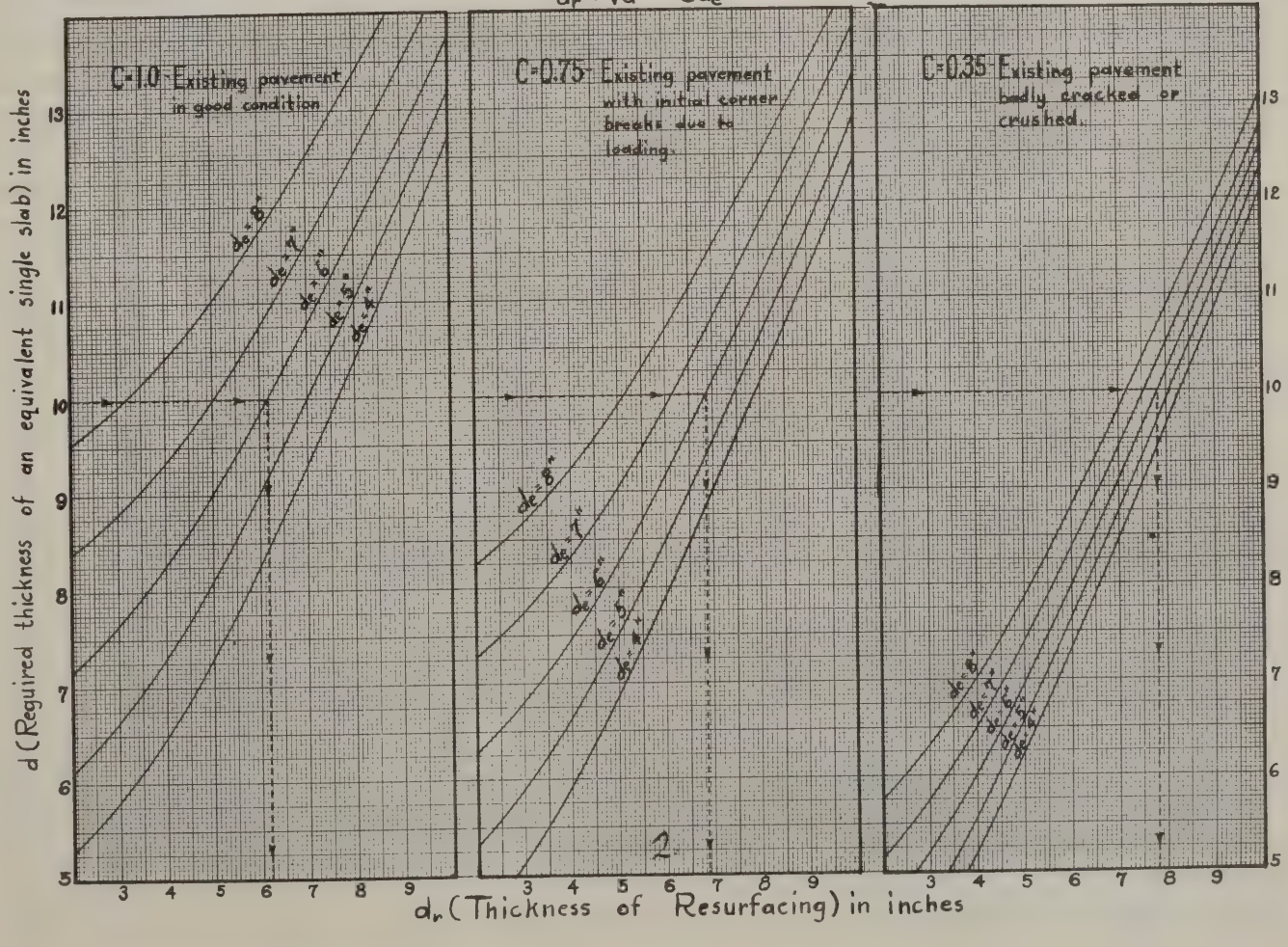


1.

Typical Section of Concrete Resurfacing
for old Concrete Pavements
from 16' to 22' widths

Design Chart for Concrete-Resurfacing for Roads & Streets

$d_r = \sqrt{d^{1.87} - Cd_e^2}$



Forms: If the old surface is to be resurfaced full width, or widening is proposed, the placing of forms for the concrete resurfacing is simplified in that the form will be on the shoulders of the old road and pinning will be no problem. However, every care should be exercised to assure that the forms are so set that movement either horizontal or vertical will not occur. If it is necessary to set one line of forms on the old concrete pavement, an offset line of holes for form pins should be drilled and then the forms set in place using offset brackets. The use of the offset brackets will allow for horizontal adjustment of the forms to get a straight line.

Joints: As previously pointed out, it is not necessary to have the same number of expansion joints in the resurfacing as are in the old pavement. However, if it is desired to provide expansion joints over the old expansion joints, the wooden header, or filler, should be cut to the crown of the old joint filler that is in place. Care should be used to secure positive assurance that the joint filler is full depth, full thickness, and full width, and that there are no holes or cracks that will permit the mortar to flow through and ruin the effectiveness of the expansion joint.

Contraction joints should be provided at the twelve-foot to fifteen-foot intervals. If the pavement is being widened, the dummy groove for the contraction joint should be extended full width of the widening for a minimum depth of one and one-half-inch. The groove should be a depth of at least one-inch on the resurfacing part of the pavement. If the corrugated joint is being used for the contraction joint, it will be satisfactory to extend the same depth of joint across the widening as is used on the resurfacing. Every care should be exercised to keep the joint vertical and straight. If sawed joints are used for the groove for the contraction joint, the sawing should be done as soon as possible and not at a time when the concrete is in maximum stress.

Subgrade: The existing pavement should be thoroughly cleaned and wetted. Cleaning should be done to the extent as previously outlined. The wetting should be of sufficient amount to assure that the old pavement is cooled down to the temperature of the mix, or as near as possible to the mix temperature. This is to avoid quick expansion or contraction of the old pavement caused by placing the new concrete on it, which could easily cause the new surface to crack while it is still green and weak structurally.

Mixing and Placing: The mixing, placing, strike off, finishing, and curing of the concrete resurfacing is the same as for regular concrete pavement. It should, however, be well cured and not allowed to dry out and contract before it has developed in strength to resist tensile stresses.

Resurfacing with concrete pavement is not new. It is an old and tried and tested construction procedure that has proven satisfactory over many years. Not enough thought is being given to this type of resurfacing at the present time. This is a type of construction that can economically provide the additional strength and width needed by present old pavements to adequately carry the larger and heavier loads of today's traffic. In addition to its desirable load-carrying characteristics, it is tops in providing smoothness and unexcelled visability.

Resurfacing with concrete pavement is adaptable to old concrete pavements, old flexible-type base with bituminous surface (providing the base is of good quality material), and old brick pavements. Its use will add years to the life of present roads that are now overloaded. Let's look into its possibilities.

A NEW DEVELOPMENT IN CONCRETE PAVEMENT CONSTRUCTION

Ted L. Edwards, Paving Engineer
Portland Cement Association
Austin, Texas

My assigned subject is, to me, a timely and significant topic for our discussion today. We are all aware of the fact that new developments are the outgrowth of new ideas that are carefully nurtured to complete fruition by men of vision and ingenuity. We have only to look around us to realize the marvelous developments that have been accomplished in many fields just in the past few years. Television, color television, jet airplanes, and many others. But let's look closely at the field of concrete pavement construction. Most of us are very familiar with the concrete paving train as we know it today, and with the exception of some refinements, it is essentially the same paving train of equipment we used in the late twenties. Of course, it is true we have improved the equipment and enlarged it too. It is much more efficient today than it was 25 years ago, but that is not what I mean. I'm talking about new ideas -- new methods that will accomplish equal or better results more efficiently and more economically.

Let's look back over those 25 years. Many of us have been building pavements that long. What progress have we made in construction procedures in that period? What new methods have been introduced to increase efficiency and reduce initial construction costs? Equipment refinements -- yes -- but maybe we have added piecemeal refinements just to keep from doing some serious thinking about new developments. We are all aware that concrete pavement

possesses outstanding qualities of durability, safety, low maintenance, and low annual cost that are extremely desirable. Clearly then, if these qualities can be preserved with greater economy of construction, we have made progress.

A new development is in the making. It is a new type of concrete paving and finishing machine now being referred to as a traveling form paver. Some of you may be familiar with some work that was done in Iowa in 1949 with a machine developed by James W. Johnson. This machine was purchased by Blaw-Knox for further development. In 1953, a similar machine was constructed by General Paving Company of Champaign, Illinois, for some work in that state. Both of these machines constructed lane-at-a-time or half-width pavements.

Another pavement was constructed in 1953 in Illinois with a traveling form paver developed by the Regenhardt Construction Company of Cape Girardeau, Missouri. This machine was built to construct full-width pavement 24 feet wide and 9-inch uniform depth.

Since that project, another contractor, Quad City Construction Company of Rock Island-Moline, Illinois, has constructed another machine much lighter and reportedly much more economical and efficient. I observed this machine in operation on U.S. 150 near Bloomington, Illinois. Incidentally, while I was there, I learned that yet another machine was under construction by Arcole Midwest Corporation of Chicago. This last machine will begin operations on a highway project near Chicago about the middle of September. Actually contracts have already been awarded and it is entirely possible that the Regenhardt, the Quad City, and the Arcole machines will all be working on highway projects in Illinois at the same time.

In the course of my observation of the Quad City machine near Bloomington, I took a few pictures including some movies that I would like to show you. It is not a long movie, I promise you that, for after all, the machine is only 26 feet wide and 39 feet over-all in length and you can't make a very long movie from such a short subject. You will note that Arcole Midwest Corporation is the contractor. The Quad City machine is being rented for this project. Arcole's new machine was not yet available.

Before we start the film, I'd like to give you some facts I learned about the machine and this particular job.

1. The concrete slab is 24 feet wide and 9-inch uniform depth.

2. The concrete slump range is two and one-half inches to three and one-half inches with about 4 per cent entrained air.

3. The entire crew for mixing, placing, and finishing consists of one mixer foreman, one mixer operator, one finishing machine operator, two finishers, and four common laborers. Nine men are all that are needed for the complete operation. A mechanical spreader would eliminate the two laborers doing the puddling in front of the strike-off screed.

4. The machine is constructed in sections with flexible connections for ease of maneuverability on horizontal or vertical curves. In the movie, only two sections are shown -- the main unit and the straight edge of finishing unit. The unit containing the longitudinal float has been omitted because this happens to be a concrete base job and the tolerance is three-eighths inch in ten feet instead of the concrete pavement surface requirement of one-eighth inch in ten feet.

5. Average production is 1,300 feet of pavement per day.

6. One other thing that I think is very interesting. I was informed that this machine represents a total investment, by the contractor that built it, of $6,000. That almost floored me, but you take a look at it and see what you think.

(Movie shown here.)

Now I am confident that a number of you have questions in mind that you would like to have answered. Some I can probably answer, others I am sure I cannot, but will try. However, before we get into a round-table discussion, let me mention a few items.

From the movie and from your experience, it is readily apparent that the subgrade preparation is a major factor in the riding quality of the final pavement finish. That

is entirely correct. As mentioned in the movie, that tractor drawn subgrader was to be replaced the following week with a self-powered unit similar in construction to the paver, but with positioning controls that would assure smooth, firm support for the paving machine. A number of equipment manufacturers now have precision subgraders that can easily be modified to the requirements of the traveling form-paving machines.

Another question may be "What about patents?" My answer is "I don't know." The Blaw-Knox Company bought the Johnson machine and I am told has applied for patents in the United States and nine foreign countries. I believe that the Regenhardt firm has applied for a patent. I do not know what action the General Paving Company, the Quad City Construction Company, or Arcole Midwest Corporation have taken. It appears they got the idea, used a little imagination, a lot of ingenuity and came out with a traveling form paver. Nor do I know how many other machines may be in the development stage now, if any.

It is certain though that contractors and engineers alike are becoming more and more interested in this new type of machine.

We all know that the cost of any pavement is determined primarily by four items: materials, labor, equipment charges, and overhead.

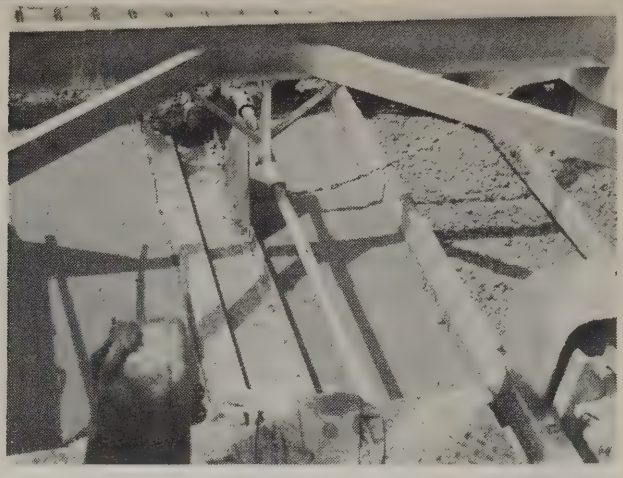
Material costs are determined by current market conditions and transportation costs, so not much can be done to reduce them or the overhead charges either. So to effect any economy of construction we must look to a reduction in equipment and labor costs.

The traveling form paver promises just such economies, at the same time providing high-quality, durable, low-maintenance, smooth-riding concrete pavements. To date, it has been reported that unit bid prices have been reduced from 10 per cent to 30 per cent by its use. Further development will probably stabilize prices somewhere in this range - and there will be further development. Almost every day these machines operate, they have visiting engineers and contractors from all parts of

This traveling form paver was constructed by the Quad City Construction Company of Rock Island, Illinois. The concrete slab is 24 feet wide and nine-inch uniform depth.

The machine operator looks ahead at the prepared subgrade with no side forms. On this project the machine is steered to follow the alignment of the metal center joint.

Concrete is supplied by the conventional 34-E Dual Drum Mixer and multiple batch trucks.



Looking down from the operator platform the strike-off is at the extreme right, then the vibrator, next the oscillating screed, and the concrete surface ready for the scraping straight edge.

Behind the burlap drag on the back side of the machine the slab presents a very nice appearance.

the country. Some serious thinking is taking place -- a new development is in the making.

To attain the greatest economy and efficiency of this type machine, it is readily apparent that pavement specifications must be tailored to its use. The following requirements would simplify its operation and reduce costs, yet provide high-quality concrete pavements:

1. Pavements should be constructed full width and of uniform thickness, without thickened edge.

2. Expansion joints should be eliminated and all longitudinal and transverse contraction joints should be sawed joints, dummy groove, or the Texas corrugated contraction joint.

3. All steel should be eliminated except the deformed tie bars necessary for holding the slabs together across the longitudinal joint.

4. To prevent slumping or lateral spreading of the fresh concrete, it is essential to maintain the closest possible control of the slump and to keep subgrade from being too wet or too dry.

5. Concrete should contain between 3 and 6 per cent of entrained air to increase its cohesiveness, reduce bleeding, and improve placeability and durability.

6. The subgrade should be carefully prepared as to grade and contour and then should not be disturbed by trucks or other vehicles.

7. The mixer should operate on the shoulder and be capable of supplying 125 cubic yards of concrete per hour.

8. Only straight or slightly curved sections of pavement should be laid with the machine. Sharp curved sections should be laid by other methods until the machine has been developed to lay short radius curved sections.

9. Curing compound of proven quality should be applied automatically and immediately after final finishing.

10. Joints should be neatly filled so as not to impair the appearance and riding qualities of the surface.

Even on superelevated curves the traveling form paver does an excellent job.

Here is a close-up of the fresh concrete slab just after the machine has passed.

Some of the mechanical improvements contemplated by the developers of traveling form machines are:

1. Addition of a device for automatically spreading the fresh concrete evenly between forms after it has been deposited on the subgrade by boom and bucket.

2. Improved vibrating action to insure compaction of the concrete from the top surface to the subgrade. This will insure that the full flexural strength corresponding to the nominal thickness of the slab and concrete used will be realized. Vibration should start automatically with the forward movement of the machine.

3. Addition of an automatic belt finisher for the final finish desired before texturing the surface with a burlap drag, if so desired.

4. Addition of a device for automatic spraying of membrane curing compound. This should be arranged to apply the curing compound uniformly and in the amount needed to prevent moisture loss. Spraying should start automatically with the forward movement of the machine.

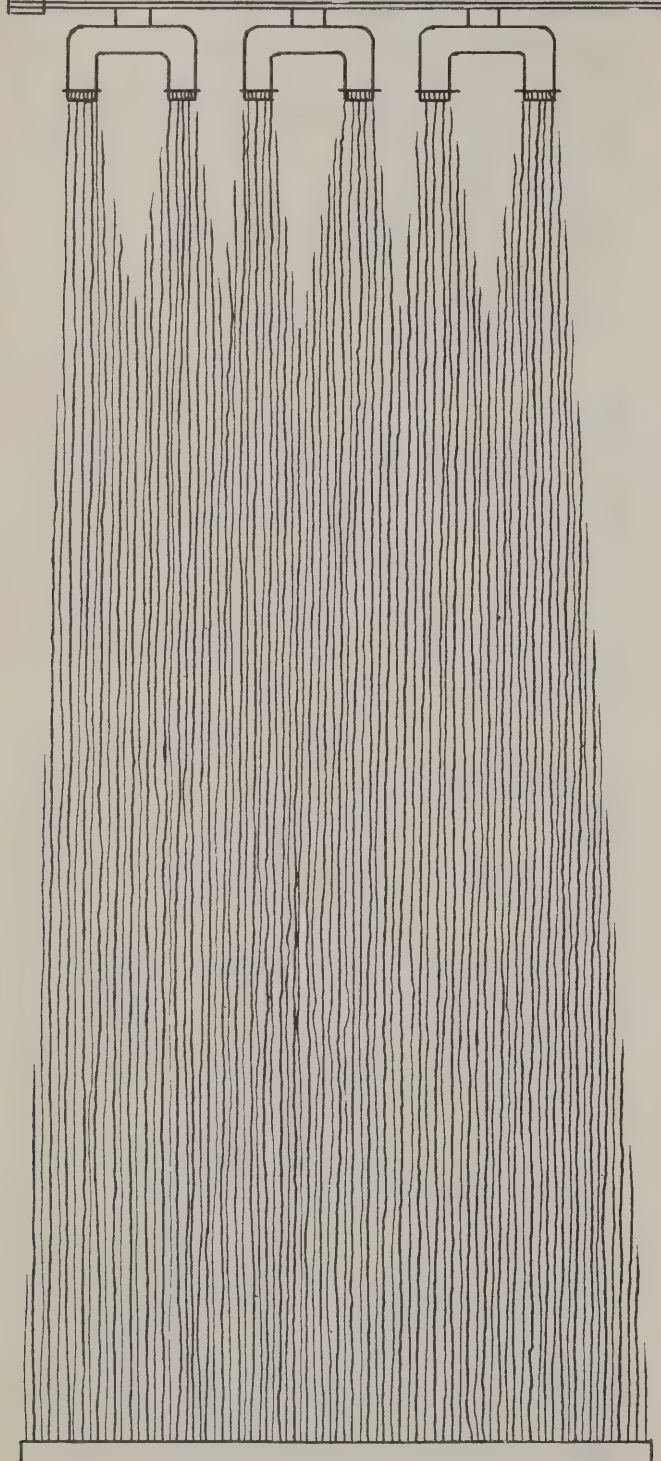
5. Replace the gas engine power unit presently used to drive the various components of the machine by means of gears, shafts, chains, and sprockets with a gas-electric generating unit and individual motor drives.

6. Develop an economical method of preparing a smooth subgrade at exact grade desired. This is a major undertaking and very important for the successful operation of traveling form pavers.

Traveling form pavers can be constructed at relatively low cost and they will also reduce the capital investment and equipment charges of a paving operation by eliminating a number of auxiliary machines and large quantities of side forms used on conventional paving jobs. The elimination of these auxiliary machines and side forms will materially reduce the present high moving-in charges. Reduction in capital investment for equipment and lowering of moving-in charges should make it possible for more contractors with limited capital to build concrete roads with traveling form pavers.

New Uses for

HVMS EMULSIONS



J.C. Roberts, District Engineer
Texas Highway Department
Abilene, Texas

The question as to whether the uses and procedures of high viscosity medium set emulsion asphalt are new is a question which we are unable to answer; therefore, it will be the purpose of this paper to describe the methods and procedures, and also the purposes for which it is now being used in District 8 for the Texas Highway Department.

It has been our observation for many years that the use of MC-1 or other cutback-asphaltic materials resulted in undesirable surfaces as far as quality was concerned and also the problem of coordination on the part of the contractor of his equipment and personnel in being able to apply the MC-1 prime on the base at the proper time was one of much concern. We found that with cutback asphalt used as a prime the behavior of this material was somewhat erratic.

On dense-type bases a residual asphalt was left on the surface of the base which could be considered working asphalt, with the balance of the asphaltic material used in the surface. On the other hand, on sand and gravel bases and on materials which gave a lesser density, the cutback would penetrate into the base and leave no residue. Many of our bases are spotted, some sections very dense and others very open. After a uniform application of asphalt for penetration work over a spotted condition there would result a wide variation in the amount of working asphalt to be combined with the cover stone. The results, especially on lightly traveled roads during the winter seasons, would be ravaged by raveling on the lean sections and intense bleeding on the fat sections. This problem is particularly prevalent in West Texas where the characteristics of one base material pit are ever changing and the material is used pit run.

In collaboration with File D-14 in Austin an experimental project was set up in 1951 on F.M. 1082 in Jones County. On this project a solution of high viscosity, medium set emulsion and water was provided in lieu of the usual MC-1 prime. It was our thought that the water and emulsion solution could be applied with the usual water truck equipment which is ordinarily used on base work. This idea proved to be very satisfactory as long as the emulsion water was applied in several applications and the per cent of emulsion to water was less than 10 per cent.

On this particular project, the base material was pit run and its compaction and finishing characteristics varied throughout the pit and along the road. The concentration of the emulsion water varied from 5 per cent to 15 per cent and was applied in several applications to the end that approximately 0.05 of residual asphalt was left on the base to act as a prime. The results indicated that the base filtered the asphalt out of the water and the amount of penetration of the emulsion and water was zero, which meant that all of the residual asphalt was left in a very thin membrane on the surface of the base. This membrane did meet, for all intents and purposes, the requirements of a prime coat in that there was created an affinity for the asphalt surface to the base.

We have carefully observed this project since its completion in 1951. There has been the very minimum amount of raveling in the winter months, and there are no fat spots on the surface. Since there was no penetration of the oil asphalt into the base, it is evident that all of the asphalt applied is now working with the cover stone which was designed in keeping with the amount of asphalt placed.

Another advantage in the use of this particular construction is simply a matter of procedure. It was not necessary for the asphalt distributor to arrive on the project until all of the base was finished and all of the base had been primed with the solution of emulsion and water. This, of course, is certainly an advantage from the contractor's point of view.

Traffic was carried on the base throughout the construction period. During the period between the placement of the final surface and the first application of emulsion and water, certain sections showed evidence of raveling. It was necessary only to make another application of water and emulsion in order to cure such conditions as they might develop on the project.

At the present time all prime coats and tack coats in District 8 utilize this method of construction on all penetration work. Using the same materials and same percentages of emulsion and multiple application of water with water trucks, a similar treatment was provided in the plans and specifications for U. S. 84 in Scurry County, north of Snyder.

On this project the plans provided for a flexible base overlay over a low asphalt surface and 125 pounds per square yard of hot mix asphaltic concrete -- Type D. Traffic volume varied from 12,000 vehicles per day within the city limits of Snyder to approximately 2,000 vehicles per day on the north end of this thirteen-mile-long project. This traffic consisted of heavy oil field trucks, mixed with the usual automobile traffic. In the prosecution of this work, the contractor crushed his material, hauled the base to the road, processed it in the usual manner of compaction and finished and applied multiple applications of emulsion and water immediately behind the finishing operation. The entire thirteen-mile section

was completely finished and treated with the 5 per cent solution of water and emulsion to the extent that approximately 0.05 of residue per square yard was left on the surface and under the wheels of traffic. This treatment carried the above volume of traffic for 47 days during the months of November and December without the necessity of refinishing any of the base.

The hot-mix paving operation was started at the end of the 47 days and we found that a tack coat was unnecessary ahead of the asphaltic concrete.

It is our opinion that the procedure outlined above, using emulsion and water as both a prime and a tack coat, effects a considerable saving in the actual cost of construction of such projects, and at the same time gives to the traveling public as good a traffic facility prior to the placement of the pavement as they have with the ultimate pavement in place.

In the fall of 1952 it was evident that some of our farm-to-market roads, both old and new, where the surfaces consisted of single asphalt surface treatment using a grade three aggregate, were deteriorating due to raveling on one side and cracking on the other. It was imperative that something be done or there would be no aggregate left on the surface and the wetting and drying and freezing and thawing cycles during the winter would cause numerous base failures. From our experience on construction projects where we utilized emulsion and water solution, we concluded that multiple treatments of the solution on old surfaces might stop the raveling and might also seal the cracks in the surface. With this in mind, the maintenance forces started utilizing the water and emulsion treatment on our highways.

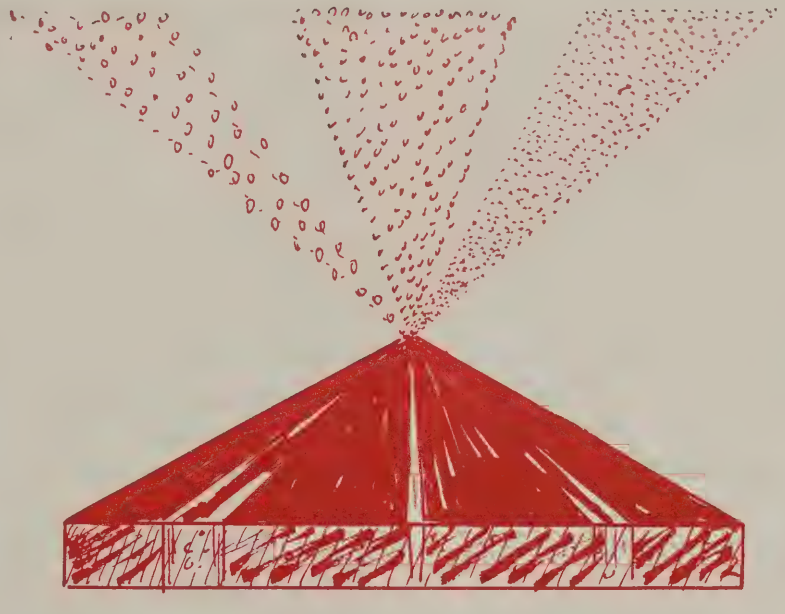
Using the multiple-treatment method, approximately 0.05 of the residual asphalt per square yard was placed. Raveling of these pavements was stopped in the middle of the winter. The many different types of cracks were also sealed.

It was observed by the engineers inspecting the surface of these roads that the roads were not only saved but improved to such an extent that seal coats which had previously been programed for them could be deferred. Of course, the size of the aggregate and depth of embedment into the membrane of the asphalt is the determining factor as to whether seals of emulsion and water without aggregate can be applied to any paved surface. Since this experience, the above-described treatment for asphalt surfaces in need of a seal coat is now being used extensively in this district by our maintenance forces in each county.

Incidentally, in those cases where a very light tack is needed on old pavements preceding hot-mix asphaltic concrete, an application of emulsion and water has been used successfully for this purpose also.

No doubt engineers in all areas have been confronted with the problem of how to better coordinate the construction operation from the time the base is finished until the time the ultimate pavement is placed. Cases have arisen where the engineer felt that a very light application of asphalt added to the surface would be of benefit to the road.

No doubt traffic problems through construction periods have presented a bothersome period to an engineering organization. To our way of thinking, the procedures outlined above permit the application of residual asphalt in very minute quantities -- where and when needed -- inexpensively, and with a common water truck.



DESIGN OF HOT MIX PAVEMENTS

J.R. Harris, Supervising Field Engineer
Texas Highway Department
Austin, Texas

An asphaltic concrete pavement mixture is a combination of granular particles, usually called mineral aggregates, with a more or less viscous liquid, asphalt. The design of an asphaltic concrete mixture means selecting the type, amount, and gradation of the mineral aggregates, and then determining by test and by calculation the amount of asphalt to use. If this combination does not meet the required provisions of the specifications, then it will be necessary to redesign the mixture, using different amounts of the same aggregate or other aggregates and determining by test, the amount of asphalt to use with this combination of aggregates. This process is repeated until a mixture has been established which will meet all of the requirements of the specifications. In most instances the first mixture will be satisfactory; however, there have been cases when as many as ten or more mixtures were designed before a satisfactory mix was secured. The exact procedure to follow in the design of an asphaltic concrete mixture is well explained in T.H.D. Bulletin C-14.

Some of the more essential properties of a satisfactory asphaltic concrete mixture are density, stability, flexibility, workability, surface texture, and durability. Some engineers are prone to stress one or more of these properties to the detriment or exclusion of the others. In order to evaluate a paving mixture, we should take the total integration of all of the above-enumerated properties. We may discuss these properties as follows:

DENSITY: Almost all paving engineers seem to agree that "other things being equal, the densest bituminous pavements are the best." I believe this viewpoint is well taken, for some of the other essential properties are almost totally dependent on this one property. Density indicates the ratio of the total volume of solids in a compacted specimen to the total volume of the specimen. When the compactive effort is held constant, such as molding laboratory specimens, the density may be varied by changing the asphalt

content. After the asphaltic concrete has been placed on the road, the density developed in the pavement is dependent on the compactive effort exerted while the mixture has plastic characteristics.

All asphalt cement used in Texas as a binder for hot-mix asphaltic concrete pavement is a viscous liquid at a temperature of 140°F. or higher. The asphalt ceases to be a viscous liquid at a temperature somewhere below 140°F. This is an important point to remember when placing bituminous mixtures in cold weather, because the mix soon becomes chilled to the point where additional rolling does not develop additional density; and, if sufficient density has not been developed, the surface will be permeable to both air and water. As a result, the surface will surely ravel when placed under present-day, high-speed traffic. I realize this condition may not be a design feature, but some engineers may consider it as such, add additional asphalt to correct the faulty behavior of the pavement, and then the following summer, experience the failures of shoving, rutting, and a slick surface so common to high densification of the pavement.

The density of a pavement should be low enough to allow for the expansion of the asphalt cement in hot weather without becoming unstable. At the same time, the density should be high enough to produce a pavement that will withstand the abrasive action of traffic, and be impermeable not only to water, but also to air, and thus tend to increase the durability of the pavement by assisting the bituminous binder to resist change from oxidation. Our specifications provide for a density of not less than 94 nor more than 98, with an optimum of 96. The minimum and the maximum values for density are not average values, so, in order to allow for normal variations in plant operations and stay within the specification limits at all times, it probably is desirable to design for a density of not less than 95 nor more than 97 as long as the stability requirements are met. Ordinarily, with given aggregates, the higher the density, the lower the stability.

STABILITY: The word stability is used in many fields of endeavor and has so many different shades of meaning that it has come to be little more than a descriptive term, such as "good" or "permanent." However, those interested in asphaltic concrete usually know what they mean by the term "stability," and as we are talking about plastic mixtures, stability means the ability to resist lateral deformation under a vertically applied load. Stability values are expressed in percentages and range from zero to one hundred. Zero stability represents a liquid which will transmit laterally the full amount of vertically applied load. A stability of one hundred represents a solid body which will not transmit laterally any portion of a vertically applied load.

During the early stages of road development, the lack of stability in black top pavements probably was the primary or decisive factor which resulted in the extensive use of a rigid type of pavement. It is a well-recognized principle that the ability of an asphalt pavement to carry loads without deformation is dependent mainly upon the internal friction of the mineral aggregate particles which constitutes the entire framework and the major portion of absolute volume of the structure. In other words, the stability of a pavement depends primarily on the mineral aggregate, and of this stability, the course aggregate (plus ten mesh size) plays the predominating role. Another well-known principle is that angular particles having rough surfaces will develop more stability than round particles with smooth surfaces. This is true for two reasons: first, the coefficient of friction is greater for the angular particles; second, and by no means the least, the area of contact is much greater.

The function of the asphalt cement is to coat each particle of the mineral aggregate with an adhesive ductile film of bitumen and to cement adjacent coated particles to each other. With the correct amount of asphalt in the mixture and when properly compacted, the resultant pavement should be able to resist all disruptive forces, including weathering elements, and should possess sufficient mechanical stability and flex-

ibility to accommodate without distortion and raveling the shocks and stresses caused by heavy high-speed traffic under all conditions. Unfortunately, complex effects arise from the paradox that the asphalt cement operates in opposite directions with regard to mechanical stability. On one hand, in very thin films it has a cohesive action tending to increase stability; on the other hand, in thicker films it acts as a lubricant which has the reverse effect. It is this lubricating property of the asphalt cement which facilitates the compression of the hot mixture under compactive effort to give the desired density in the pavement.

It follows therefore that too much attention cannot be given to selecting the optimum asphalt content for any given project. Too low a value usually produces low density with a corresponding high stability and inevitably results in raveling and disintegration of the surface. Too high a value usually produces high density, with a corresponding low stability, and results in rutting, shoving, and a corrugated, wavy surface.

A minimum stability value of 35 has been established by the Texas Highway Department for work constructed under its supervision. A maximum stability value has not been established, because as in other road building materials, the stronger the product, usually the better. Probably the only instances in which high stabilities should be viewed with concern are those times in which the densities are abnormally low. If the inherent characteristics of an aggregate are such that a relative high stability is secured with the optimum density or slightly higher density in the mixture, I see no reason to deliberately destroy this stability by incorporating inferior materials into the mixture.

FLEXIBILITY: Flexibility is a relative term often used, but without any definite range of values having been established by the Texas Highway Department to determine the degree of flexibility desirable in a pavement. All pavements are flexible to a certain extent, including portland cement concrete. When we speak of a flexible pavement, I think of a pavement having the ability to remain in contact with the supporting foundation without cracking. When the foundation itself is a flexible or compactable base, the property of flexibility is particularly important. When the pavement is laid on a foundation having a high degree of rigidity, flexibility becomes relatively unimportant.

According to the preliminary bulletin on "Fundamentals Governing the Design of Bituminous Paving Mixtures" by Committee A-4, Highway Research Board, National Research Council, "The principal factors of a bituminous paving mixture affecting the property of flexibility are, (1) the quantity of the bituminous binder, (2) the viscosity and temperature susceptibility of the binder, and (3) the quantity of mineral filler. These three factors which affect the flexibility of the pavement also influence the stability of the pavement. It is obvious, therefore, that in designing the mixture, a balance should be struck so that flexibility will be obtained without too great a sacrifice of stability."

Flexibility may be improved without any appreciable loss in stability by reducing the amount of finer particles and increasing the amount of coarser particles. This reduces the total surface area of the particles and permits a greater asphalt film thickness. The thicker the asphalt film, the greater the resistance it will have to cracking when flexed.

Since it has been shown that stability is primarily a function of the characteristics of the aggregates, and flexibility is primarily a function of the film thickness and character of the asphalt binder, a high stability value does not necessarily mean lack of flexibility. Conversely, a high degree of flexibility usually will mean a very low stability.

It therefore follows that if a foundation does not possess sufficient rigidity to support the overlaying asphaltic concrete pavement, the proper solution to this problem would be to build more rigidity into the foundation rather than attempt to build a greater amount of flexibility into the pavement.

WORKABILITY: Workability is a relative measure and is meaningless unless considered in reference with some construction procedure. In view of the development of

efficient modern asphaltic concrete spreading and finishing equipment, the workability of bituminous mixtures is not of as great importance as it was when hand-placing methods were used. A mix is workable when it is easy to spread and rake and it should not be displaced by the roller. A mixture having crushed angular particles requires more rolling to produce the desired density in the pavement than does a mixture having round smooth particles. This may be as good a time as any to state that when placing asphaltic mixtures on new bases, particularly where traffic was not allowed on the new base, considerable difficulty in rolling the mixture is often encountered. This difficulty is not due to the mixture but results from the base flexing under the rollers. Ordinarily, the thicker the pavement being placed, the more difficult it is to roll without displacement. Other difficulties encountered in rolling may be attributed to too high a temperature of the mixture, resulting in an unstable condition; too much sand, which creates a high degree of lubrication with a corresponding loss in stability; and an insufficient amount of asphalt, which results in a lack of sufficient cohesion to bond the particles together when pressure is applied. As a rule, low-density mixtures are harder to roll than are those mixtures with higher densities.

SURFACE TEXTURE: Surface texture is a very important integral part of a bituminous pavement that too often is neglected. It must be remembered that it is upon the surface that the wheels roll, and it is only the surface of the pavement that the public sees and uses in judging its merits. Very often a slick surface is the cause of a serious accident.

In designing and constructing a bituminous paving mixture, care should be taken to avoid an excess of bitumen on the surface of the pavement either immediately after construction or after the pavement has been subjected to additional densification by traffic and by climatic conditions. Sufficient voidage should be provided in the compressed bituminous pavement to allow for increased densification by traffic loads and for expansion of the aggregates and binder caused by temperature increase so that exuding of bitumen on the surface will be avoided. When used, the sequence of placing a tack coat should be such that the tack coat is not tracked upon the completed surface.

A tightly compacted, low-voidage surface resembling a sheet asphalt surface will soon become very slick due to the oil drip from the ever-increasing traffic count. It would then be desirable to design a mixture which, when placed, will produce a surface with a relative open texture and having nonskid properties, yet prossessing sufficient density to insure adequate durability. Our specifications provide for this type of mixture. For hot-mix asphaltic concrete pavements, it is called Type F. This type of pavement should be used when the traffic count justifies it and finances are adequate. In many instances the desirable properties of a Type F mixture can be approached with a Type D mixture without any additional cost to the Texas Highway Department by using the maximum amount of a course aggregate allowed under the specifications, and without sacrificing any part of the other necessary properties essential to a good asphaltic concrete pavement. This has been done with very gratifying results.

DURABILITY: A durable pavement must be able to retain the desirable properties which we have discussed. The longer these desirable properties are retained, the more durable the pavement. Since there will be little change in the mineral aggregate of a bituminous mixture, durability will then depend upon the ability of the bituminous binder to resist change.

Those factors which tend to assist the bituminous binder in resisting change are the density of the pavement and the thickness of the asphalt film surrounding the individual aggregate particles. The denser the pavement, the more impermeable it will be to water and air -- resulting in less oxidation of the bituminous binder. As the asphalt film is increased, the ratio of the surface area to the total amount of asphalt in the pavement is decreased, thus increasing the durability of the pavement.

GEOMETRIC DESIGN AND TRAFFIC CONTROL SECTION

Morning: Room 2-C

Chairman: Professor F. J. Benson
A & M College of Texas

10:00 Design and Operation of Climbing Lanes
Frank H. Scrivner, Supervising Research Engineer
Texas Highway Department
Austin, Texas

George C. Wall Jr., Resident Engineer
Texas Highway Department
Mineral Wells, Texas

Afternoon: Room 2-C

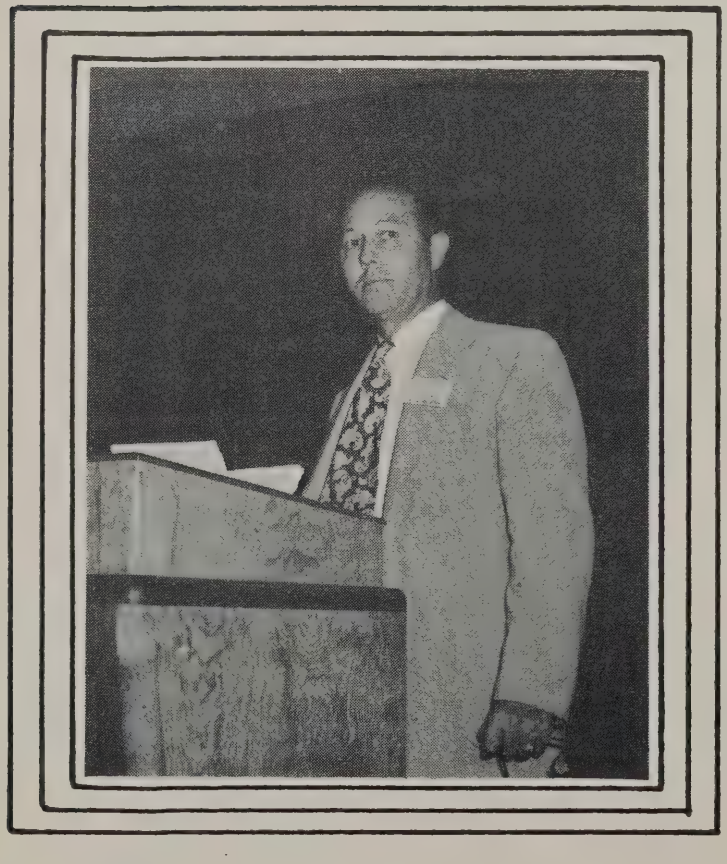
Chairman: F. E. Lovett, Expressway Engineer
Texas Highway Department
Fort Worth, Texas

1:30 Problems in Horizontal Alignment of Expressways
R. A. Bossy, Expressway Engineer
Texas Highway Department
San Antonio, Texas

3:00 Selection of Expressway Pavement
F. A. Harris, Senior Designing Engineer
Texas Highway Department
Houston, Texas

MOTION OF HEAVY VEHICLES ON HIGHWAY GRADES

Frank H. Scrivner, Supervising Research Engineer
Texas Highway Department
Austin, Texas



Theory: Consider a vehicle (Figure 1) of gross weight W, traveling at a variable velocity v, on a grade inclined at an angle Θ, with the horizontal, the value of Θ being taken as positive if the vehicle is ascending and negative if it is descending. If g represents the acceleration of gravity and t the time, then neglecting that part of the driving force required to overcome the inertia of rotating parts, we may write the force equation,

$\frac{w}{g}\frac{dv}{dt} = P - W \sin \Theta$, where P, a variable,

may be termed the net driving force acting on the vehicle. The above equation may be rewritten in the form

$$\frac{P}{W} = \frac{1}{g}\frac{dv}{dt} + \sin \Theta \qquad (1)$$

It is seen that

$$P/W = \sin \Theta \qquad (2)$$

whenever the vehicle is operating at maximum sustained speed on any grade, that is, when $dv/dt = 0$. Thus, by observing the maximum sustained speed of the vehicle on a variety of grades, a graph like the one shown in Figure 2 may be constructed, where the maximum sustained speed in ft./sec. was plotted against the sine of the grade angle. By breaking up this graph into a series of straight line segments, each with an equation of the type,

$$P/W = av + b \qquad (3)$$

we may substitute the right hand side of Equation 3 in Equation 1, and solve the latter with the following result:

$$x = \left[\frac{1}{a}\,\frac{v-v_o}{g} + (\sin\Theta - b)t\right] \qquad (4)$$

$$\text{when } t = \frac{1}{ag}\log\left(\frac{av + b - \sin\Theta}{av_o + b - \sin\Theta}\right),$$

where x = distance (feet) truck travels in changing velocity from v_o ft./sec. to v ft./sec., and t = time (seconds) required for truck to travel the distance, x feet.

The constants, a and b, are obtained from the appropriate straight-line segment of the graph of P/W versus v (Figure 2 or Figure 8), a being the slope of the segment and b its intercept on the vertical axis of the graph. Where more than one segment of the graph is involved in the velocity change, then increments of time and distance must be computed separately from Equation 4 for each segment, and added to obtain the required time and distance.

The characteristics of the design truck now being used in climbing-lane design are indicated in Figures 2 and 3. Figure 3 was an enclosure of Road Design and Maintenance Information Circular 1-52. Equations 4 were used in preparing Figure 3, the values of a and b having been taken from Figure 2.

Road Test of A Heavy Vehicle: In December 1953 a road test was conducted by the Planning Survey of the Texas Highway Department on a section of Ranch to Market Road 93 in Travis and Burnet Counties west of Austin in an effort to provide data from which the simplified theory represented by Equations 4 could be checked or corrected, if necessary. The vehicle used was an International Harvester R-195 two-axle truck-tractor (146 net horsepower at sea level) and a 33-foot Hobbs tandem-axle flat-bed trailer, both loaned to the Department free of charge by the respective manufacturers. Table 1 gives essential data pertaining to the tractor. The trailer was loaded with steel piling, the gross weight of tractor and trailer being 57, 180 pounds. (Figure 4.)

In running the tests, pneumatic tubes or detectors which actuated electric switches when run over were first stretched transverse to the highway at 100-foot intervals on a

selected grade. Two instruments, an Esterline-Angus 20-pen graphic recorder with one-tenth-second accuracy (Figure 5) and an oscillograph and camera with tuning-fork timer accurate to 1/1000 second (Figure 6) recorded the time each of the four axles passed over the pneumatic tubes during the test.

Eleven grades ranging from 700 to 1,500 feet in length, and from 0.16 per cent to 7.62 per cent in inclination, were used in the test.

In all test runs the driver, an employee of the Department, attempted to maintain the highest possible speed while remaining within the range of engine rpm recommended by the manufacturer and marked on the speedometer.

UP-GRADE ACCELERATION RUNS: The driver approached the grade at the bottom at a very low speed (one or two miles per hour). When within three or four feet of the first detector, he accelerated as rapidly as possible and continued to accelerate until he had passed over the last detector at the top of the grade. If he had not reached maximum sustained speed at that time he returned to the bottom of the grade and repeated the run, except that he approached the first detector at approximately the speed and in the gear he had previously passed the last detector. This procedure was followed until maximum sustained speed was attained.

UP-GRADE DECELERATION RUNS: The driver attempted to approach the grade at the bottom at a speed equal to or greater than 47 miles per hour (the statewide average for trucks) and attempted to reach the top at the highest possible velocity. If, after passing the last detector, his speed was still greater than maximum sustained speed, he returned to the bottom of the grade and repeated the run, except that he approached the first detector at approximately the speed and in the gear he had previously passed the last detector. The process was continued until the velocity on the grade was reduced to maximum sustained speed.

DOWN-GRADE ACCELERATION RUNS: At the top of the grade the driver approached the first detector at one or two miles per hour, then accelerated as rapidly as possible. If, on passing the last detector, he had not attained a speed of 47 miles per hour, he returned to the top of the grade, making his approach on the second down-grade run at the speed and in the gear he had previously passed the last detector, and again accelerated as rapidly as possible. The process was repeated until he attained a speed of at least 47 miles per hour on the grade.

All told, there were 118 test runs of the types described above.

Results of Road Test: Velocities for use in plotting speed-distance curves representing the test runs were computed as follows from the basic data:

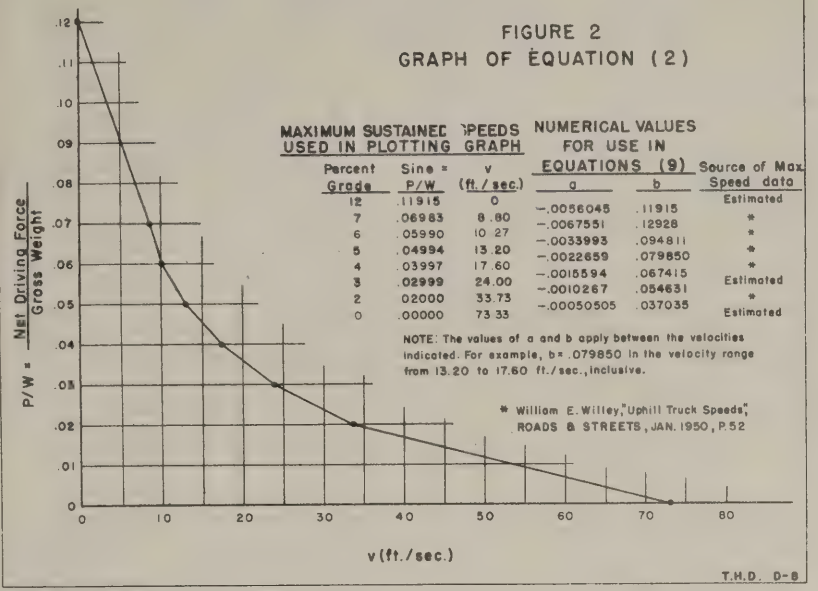
(a) From 20-pen recorder data (accuracy 1/10 second): The velocity at the instant when the front axle of the vehicle was midway between two successive detectors was taken equal to the distance between detectors (100 feet) divided by the corresponding time interval.

(b) From Oscillograph data (accuracy 1/1000 second): The velocity when the mid-point between the second and third axles was over a detector was taken equal to the distance between those axles (18.62 feet) divided by the corresponding time interval.

Simultaneous values of speed (mph), and distance from the first detector to the front axle of the truck (feet), were plotted as points on separate graphs for each of the 118 runs made on the eleven grades. A few of the resulting speed-distance curves are shown in Figure 7.

Since the grades were relatively short, it was necessary to devise a logical method for fitting together into a continuous speed-distance curve for each grade the short segments plotted from individual runs. It was also necessary to find a means for interpolating between the grades used in the test in order to plot speed-distance curves for integral values of per cent grade for comparison with the curves now in use (Figure 3). For accomplishing both of these purposes, use was made of Equations 1 and 4 in the manner described below.

FIGURE 2
GRAPH OF EQUATION (2)



MAXIMUM SUSTAINED SPEEDS USED IN PLOTTING GRAPH

Percent Grade	Sine = P/W	v (ft./sec.)
12	.11915	0
7	.06983	8.80
6	.05990	10.27
5	.04994	13.20
4	.03997	17.60
3	.02999	24.00
2	.02000	33.73
0	.00000	73.33

NUMERICAL VALUES FOR USE IN EQUATIONS (9)

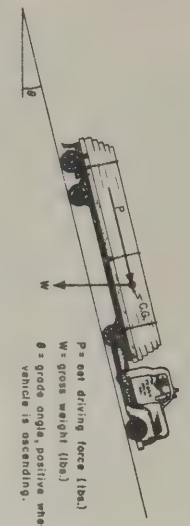
a	b	Source of Max. Speed data
-.0056045	.11915	Estimated
-.0067551	.12928	*
-.0033993	.094811	*
-.0022659	.079850	*
-.0015594	.067415	*
-.0010267	.054631	Estimated
-.00050505	.037035	*
		Estimated

NOTE: The values of a and b apply between the velocities indicated. For example, b = .079850 in the velocity range from 13.20 to 17.60 ft./sec., inclusive.

* William E. Willey, "Uphill Truck Speeds", ROADS & STREETS, JAN. 1950, P.52

T.H.D. D-8

FIGURE 1



Mass X Acceleration = Force,

or $\frac{W}{g}\frac{dv}{dt} = P - W \sin\theta$,

Where v = velocity (ft./sec.), and
t = time (sec.).

(The additional driving force required to accelerate rotating parts is neglected.)

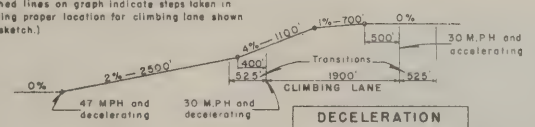
P = net driving force (lbs.)
W = gross weight (lbs.)
θ = grade angle, positive when vehicle is ascending.

FIGURE 3

SPEED-DISTANCE CURVES
ESTIMATED FOR A TYPICAL HEAVY TRUCK OPERATING ON VARIOUS GRADES

EXAMPLE OF USE OF CURVES

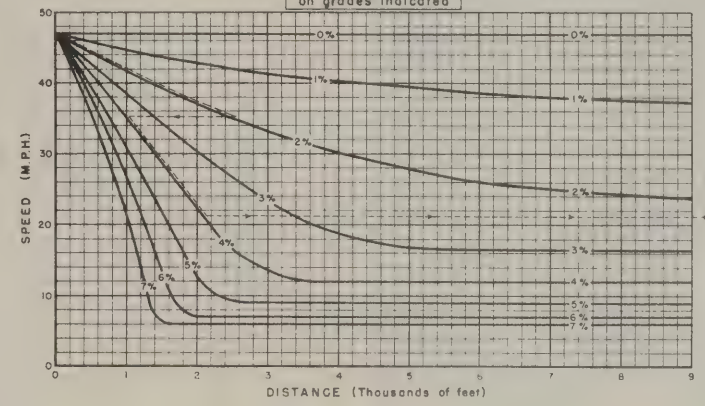
(Dashed lines on graph indicate steps taken in finding proper location for climbing lane shown on sketch.)



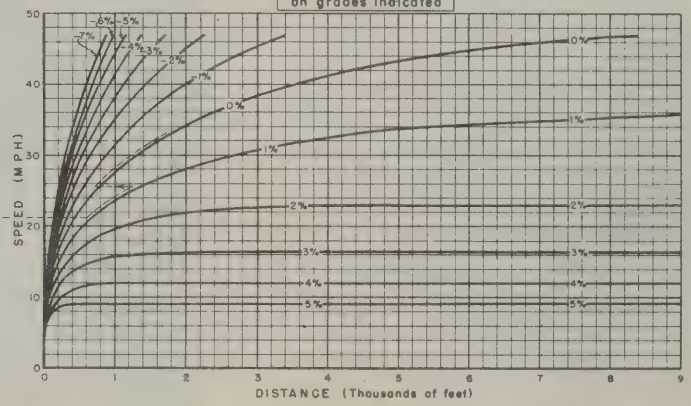
WARRANTS FOR CLIMBING LANES

CLASS B HIGHWAYS — Provide climbing lane and parking shoulder.
CLASS C HIGHWAYS — Desirable treatment: same as for CLASS B HIGHWAYS. Minimum treatment: convert shoulder to climbing lane
CLASS D HIGHWAYS — Make studies to determine feasibility of converting shoulder to a climbing lane, taking into account (1) construction costs and, (2) volume of heavy trucks.
CLASS E HIGHWAYS — Climbing lanes not considered necessary

DECELERATION on grades indicated

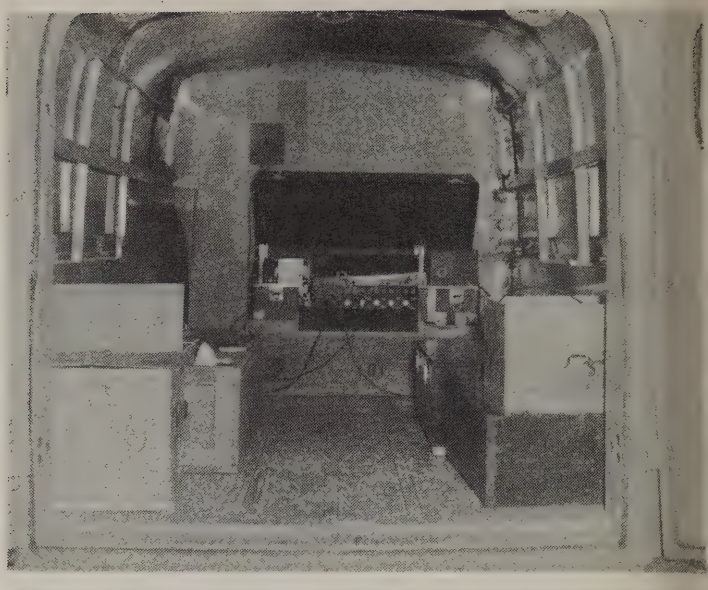


ACCELERATION on grades indicated

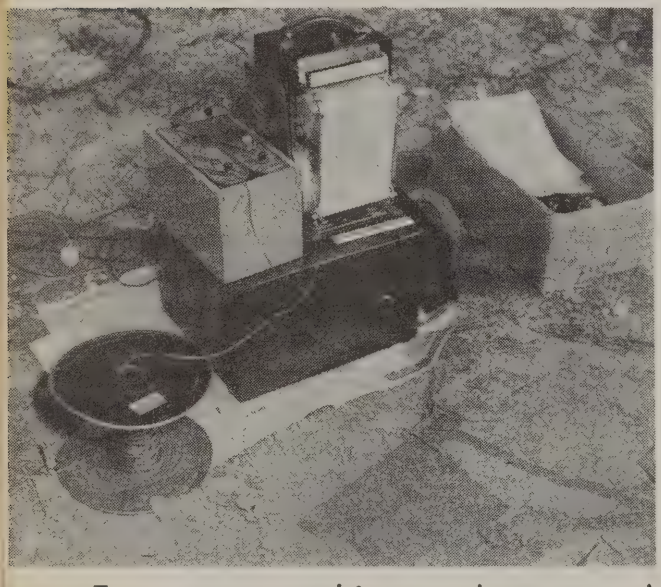


THD D-8

Vehicle used in road test. Truck-tractor is over a pneumatic detector.



Oscillograph and camera, in portable dark room, recorded time each axle passed over pneumatic detector.



Twenty-pen graphic recorder measured time required for truck to travel the 100-foot distance between pneumatic detectors.

The acceleration, dv/dt, was computed from oscillograph data for a given instant, and the grade angle Θ, being known, these values were substituted in Equation 1 and the numerical value of P/W was computed for that instant. Each value of P/W so computed was plotted on a graph against the corresponding velocity. These values are represented by the solid black points and circled points in Figure 8. Through these plotted points an average line was drawn to establish the graphical form of P/W associated with the test vehicle. The reason for the wide scattering of points on this graph is not known, but the scattering might have been due to unavoidable variations in wind direction, wind velocity, road roughness, driver behavior, and possibly in some of the time measurements themselves.

Next, from the graph of average values of P/W, and by use of Equations 4, a set of three speed-distance curves (up-grade deceleration, up-grade acceleration, and down-grade acceleration) was computed and plotted for each of the eleven test grades.

Finally, the 118 speed-distance curves previously plotted directlyy from the observed data were compared with the corresponding speed-distance curves computed by use of the graph of average values of P/W substituted in Equations 4. Fair agreement was generally obtained between the two sets of curves, and it was concluded that Equation 4 could be used in conjunction with the experimental data of Figure 8 to compute a series of speed-distance curves which would represent the behavior of the test vehicle with fair accuracy on any grade. Accordingly, the speed-distance curves of Figure 9 were computed.

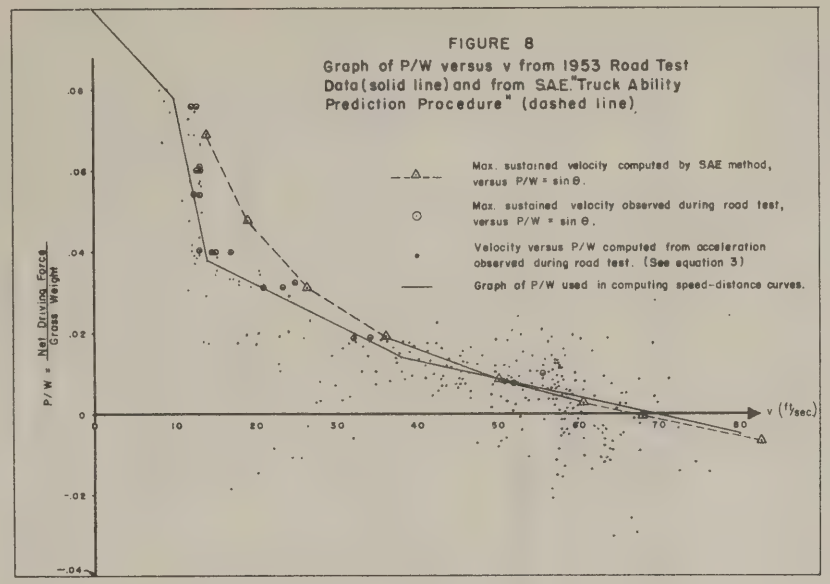


FIGURE 8
Graph of P/W versus v from 1953 Road Test Data (solid line) and from SAE "Truck Ability Prediction Procedure" (dashed line)
Max. sustained velocity computed by SAE method, versus P/W = sin θ.
Max. sustained velocity observed during road test, versus P/W = sin θ.
Velocity versus P/W computed from acceleration observed during road test. (See equation 3)
Graph of P/W used in computing speed-distance curves.
P/W = Net Driving Force / Gross Weight
v (ft/sec.)

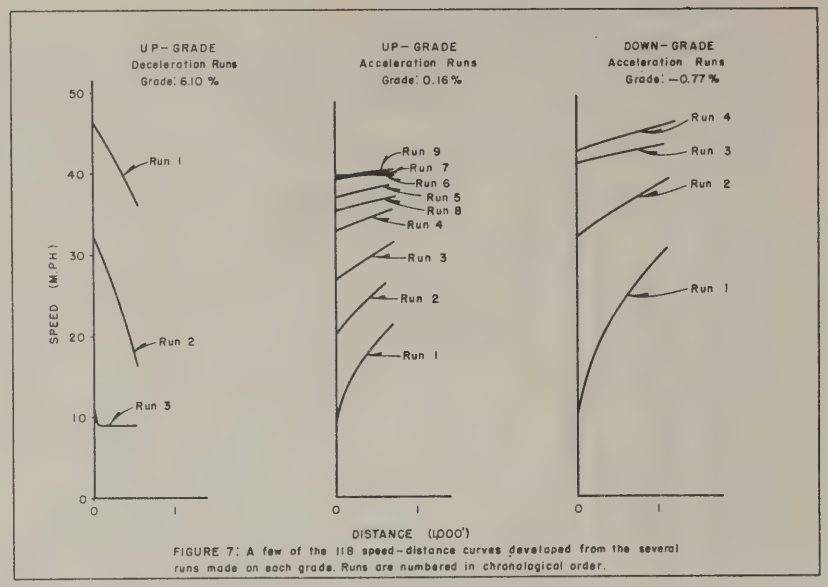
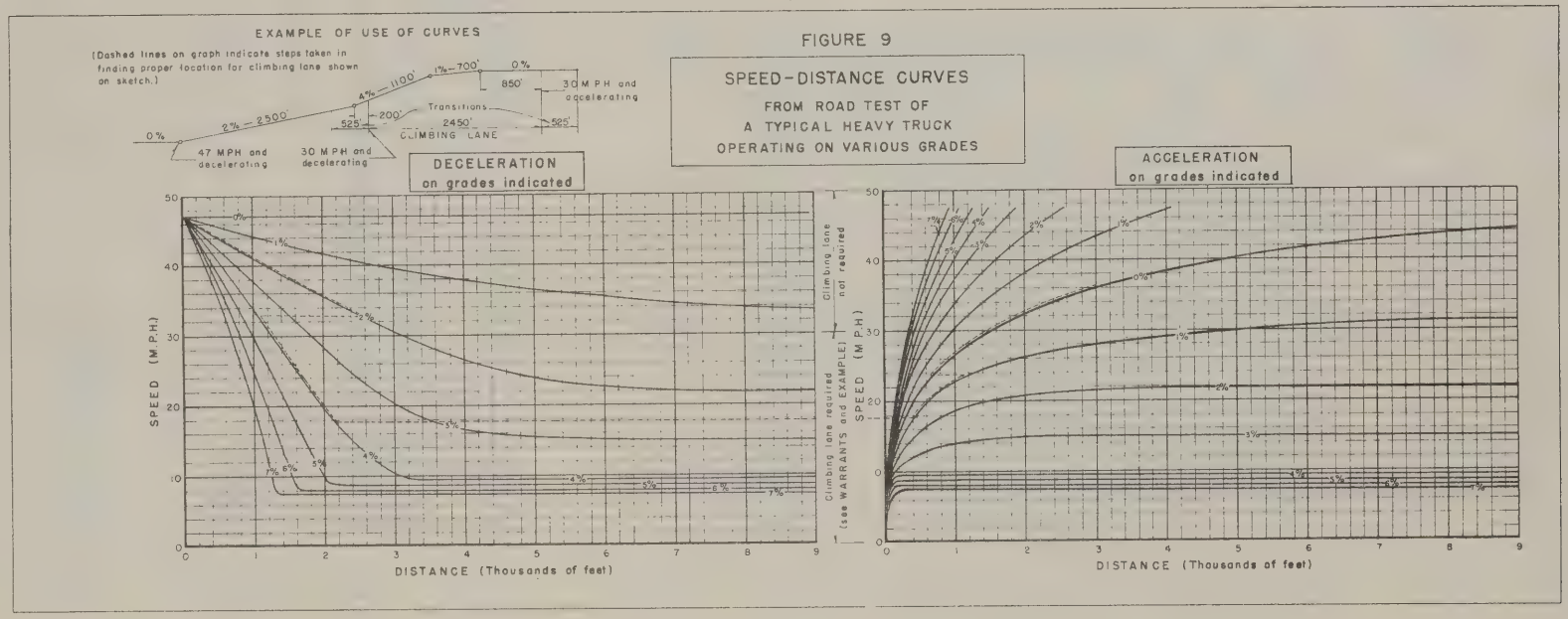


FIGURE 7: A few of the 118 speed-distance curves developed from the several runs made on each grade. Runs are numbered in chronological order.



If Figure 9 is compared with Figure 3 it will be seen that the vehicle used in the 1953 road test performed at speeds generally lower than predicted by the speed-distance curves of Figure 3. As a result, climbing lanes designed from the road-test results would be longer than those designed according to present standards.

Actually, the two sets of curves (Figure 3 and Figure 9) represent but two of the many types of vehicles which will use our climbing lanes. Many will be even faster than the relatively fast vehicle represented by Figure 3. Some will be slower than the comparatively slow vehicle represented by Figure 9. The 1953 road test having served its purpose, (which was to confirm the simplified theory of the motion of heavy vehicles as represented by Equation 4), the next logical step in the development of the theory of climbing-lane design is to determine whether the design truck now in use is too fast, too slow, or approximately correct for use in climbing-lane design. It may also be that the use of 30 miles per hour as the critical speed at which trucks should be taken out of and returned to the traffic stream may require revision. The comments of field engineers are requested on these questions.

Acknowledgements: The following Department employees were especially helpful in conducting the road test or in analyzing the data. Reed Baker and K. G. Crawford of the Planning Survey Division, W. R. Welty of the Maintenance Operations Division, Hubert Henry and S. W. Adair of the Road Design Division, and Gene Voltz of District 14. Miss Leah Moncure of the Road Design Division did the numerical work in connection with the reduction of the data and the preparation of the graphs.

For their generous cooperation thanks are also due Johnny Wright of Hobbs Trailer Company, H. T. Rosell and J. H. Gillum of International Harvester, P. J. Rudolph of the Petty Geophysical Engineering Company, and C. M. Ogle of the Texas Motor Transportation Association. The maximum sustained speeds shown in Figure 8 were computed from formulas furnished by Fred B. Lautzenhiser, Chairman of the SAE Subcommittee on Classification and Evaluation of Transportation Engineering Formulas.

TABLE I

DATA PERTAINING TO TEST VEHICLE AND CONDITIONS OF OPERATION

1. Vehicle indetification - International R-195 Tractor.
2. Vehicle over-all maximum dimensions (a)Height - 7.75', (b)Width - 7.75'.
3. Total gross weight - 57,180 pounds.
4. Manufacturer's maximum gross vehicle weight rating - 50,000 pounds.
5. Gear ratios: (a)Transmission - 6.98, 3.57, 1.89, 1.00, 0.825 (overdrive). (b) Aux. trans. - none. (c) Axle - 6.5, 8.86. (d) Total gear reductions - 61.84, 45.37, 31.63, 23.21, 16.75, 12.28, 8.86, 6.50, 7.31, 5.36.
6. Tire size - 10.00 x 20.
7. Net engine power at sea level - 146 hp at 2,600 rpm.
8. Altitude - 950 ft.
9. Road service type and condition - bituminous, good.

THE OPERATION OF A CLIMBING LANE

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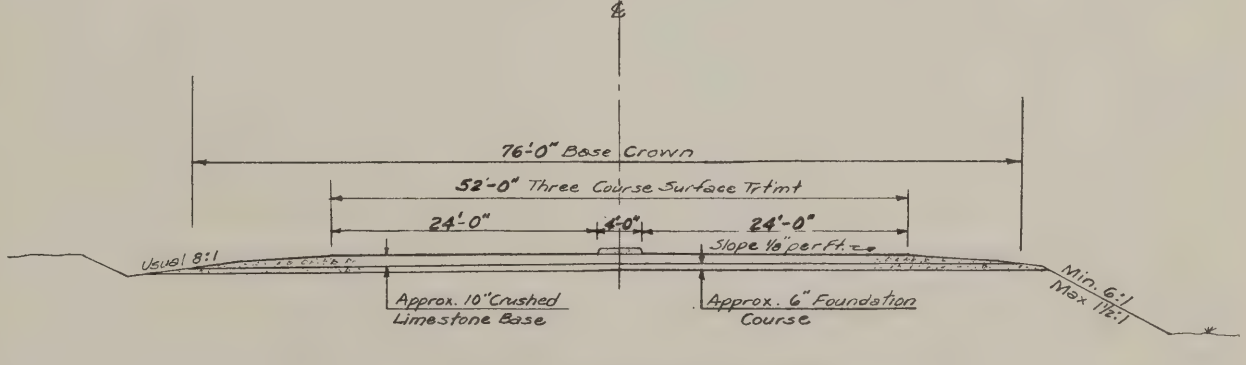
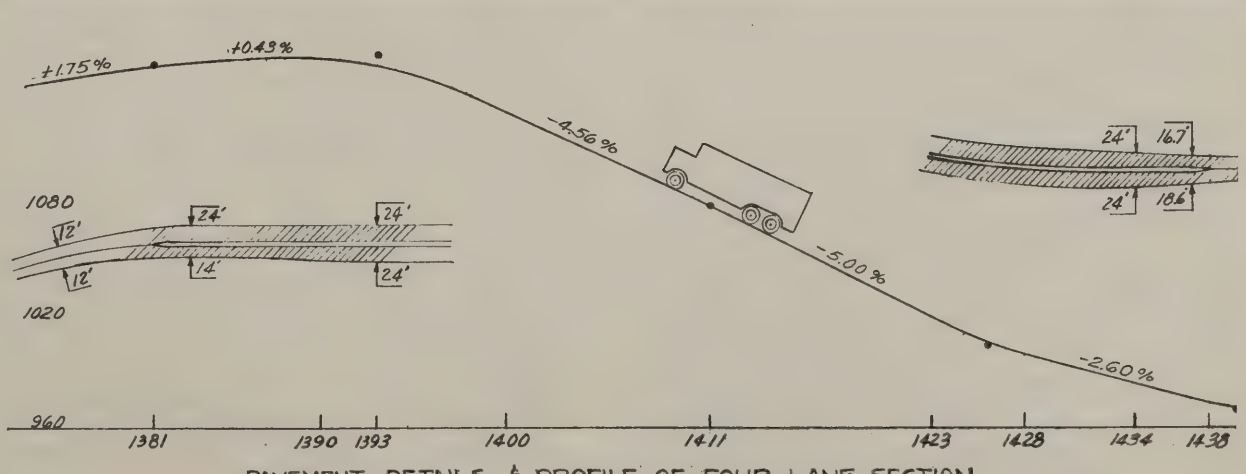
Having only one climbing lane in our residency, the scope of these operational observations is necessarily limited to a particular section of steeply graded four-lane roadway, the north lane of which serves as a climbing lane. This section of roadway occurs on U. S. 180 in Palo Pinto County and begins approximately 4.1 miles west of the Brazos River Bridge.

Due to the fact that the six-mile project in which this climbing lane is included was planned and under construction at the time the present-day warrants for climbing lanes were issued, this section has its entrance too far downhill for conformance with the present-day criteria. Since construction of the entrance had been completed, it was left in its lengthened state. However, at the planned exit, design truck speed was a mere eighteen miles per hour. The exit was under construction, so it was lengthened by field change to bring the design truck speed to the required 30 miles per hour. Therefore this north lane, as constructed, meets all minimum geometric requirements for a climbing lane.

As shown in Figure 1, this four-lane section is constructed on a 76-foot base crown with 52 feet of three-course

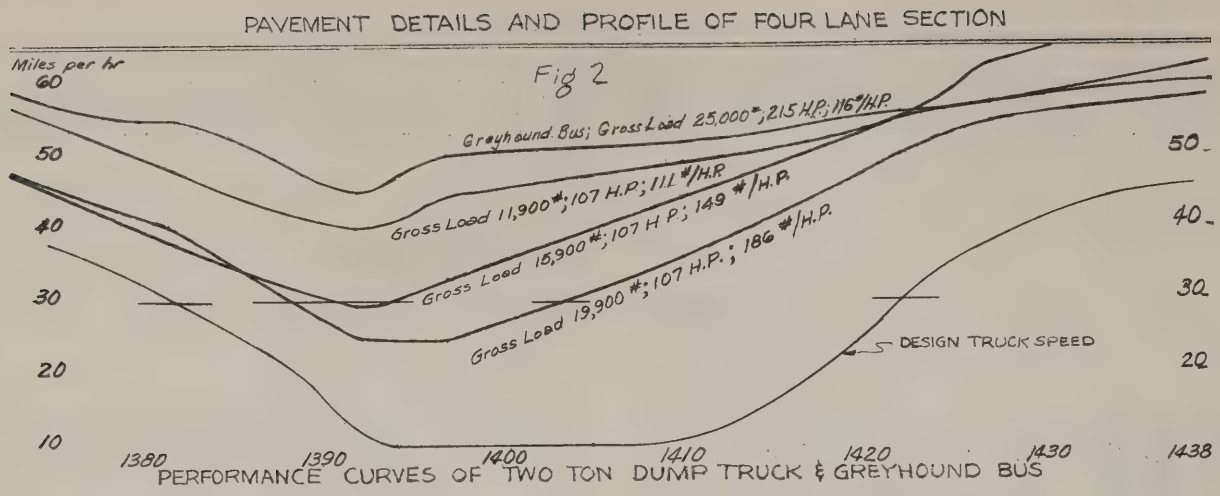


TYPICAL FOUR LANE SECTION



PAVEMENT DETAILS & PROFILE OF FOUR LANE SECTION

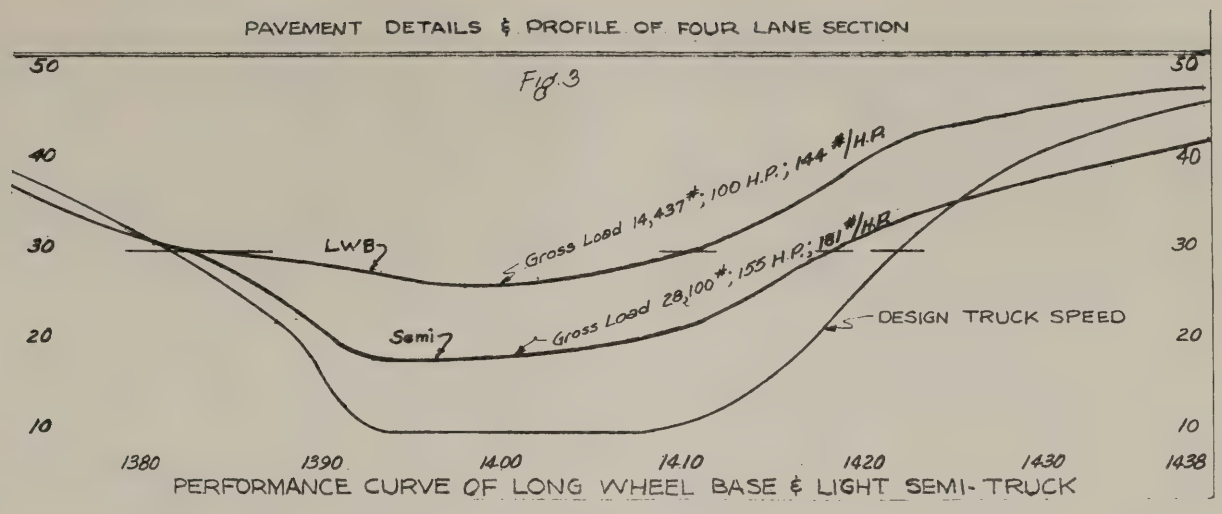
Fig. 1



surface treatment. This three-course surface treatment is divided by a four-foot concrete median, leaving two ascending lanes of twelve feet each and two descending lanes of the same widths. Ascending grades proceeding in a westerly direction are 2.60 per cent for 1,350 feet, 5 per cent for 1,500 feet, and 4.56 per cent for 1,800 feet. The vertical rise over this steeply graded section is approximately 190 feet. Entrance and exit transition distances are 700 feet respectively.

Various local truck distributors were contacted for specification data concerning rear-axle ratios, transmission ratios, and the size engines to be expected in their trucks. Here it was found that in any given truck one might find any one of three rear-axle ratios and a choice from as many as eight different engines. Five forward-speed transmission were in general standard for a given make and model. Since these five forward speed transmissions, coupled with two-speed rear axles, make it possible to select from ten different engine speeds for one turnover of the rear wheels, it becomes a difficult task for even a driver who is familiar with this section to shift at the proper time and to the proper gear in order to obtain maximum performance from his truck. This is graphically illustrated by the performance curves of the various trucks tested when their speed is plotted against their distance up the hill. Acceleration and deceleration on any given series of grades depend upon a number of factors among which are tractive ability, transmission ratio, rear axle ratio, torque, pounds carried per horsepower, and last but by no means least, driver characteristics. In order to have a uniform basis of comparison for the trucks tested, pounds per horsepower was chosen.

To begin our operational observations of this climbing lane, a new two-ton maintenance dump truck weighing 7,900 pounds empty was successively loaded with two tons for a gross load of 11,900 pounds, four tons for a gross load of 15,900 pounds, and six tons for a gross load of 19,900 pounds. With the 107 horsepower engine of the dump truck, these loads compute to be 111, 149, and 186 pounds per horsepower respectively. This truck was rated at 16,000 pounds gross weight and was loaded in each case with sacks of cement and salt. (Figure 2.) With each load six runs were made up the hill with driver instructions for three of the runs at each loading to get the maximum performance possible from his truck until reaching station 1361. For the other three runs at each loading, the driver was instructed to limit his truck speed to 46 miles per hour at station 1438. This speed and this location were chosen because the design truck speed is 46 miles per hour at this point. The performance curves for these two different methods of climbing the grade took a slightly different shape for each loading and showed a slightly lower speed for the limited performance up to the 19,900 pound gross load. For this loading



after the minimum speed of twenty-four miles per hour in each case the performance curves throughout the acceleration range were very nearly coincident. Limited performance was not plotted, however, since a restriction had been placed on the driver and they were felt not to be representative. With the two-ton pay load (111 pounds per horsepower) the dump truck entered at 65 miles per hour, dropped to 40 miles per hour, and passed through the exit at 49 miles per hour. The addition of two tons pay load for 149 pounds per horsewpoer brought minimum speed to 29 miles per hour for a net drop of eleven miles per hour. However, a further addition of two tons, making the ratio 186 pounds per horsepower, dropped the minimum speed only five miles per hour from the previous loading for a low speed of 24 miles per hour. It will also be noted that for this series of grades it was necessary to overload the dump truck by 3,900 pounds, or to load it to approximately 125 per cent of rated capacity to attain a constant minimum speed on this series of grades. At this weight, the overload springs were flat and the load was high enough to be in danger of slipping over the tail gate as the truck mounted the grade. For these reasons the tests were halted at this point. The conclusion to be drawn here is that since all curves are above that of the design truck, dump trucks do not control the beginnings or endings of climbing lanes but do attain low enough speeds to be considered in discussing climbing lanes.

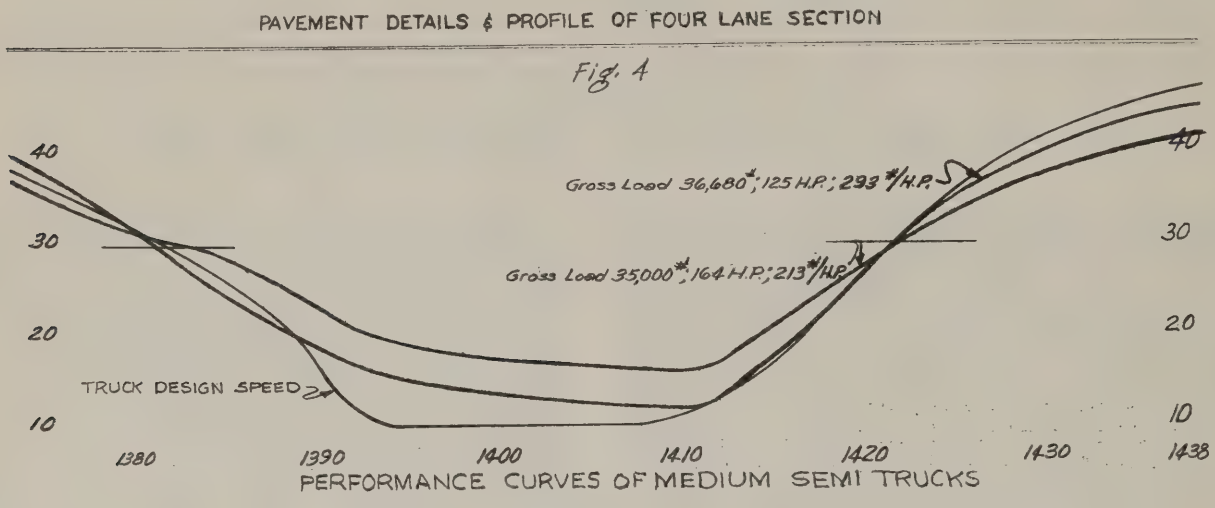
During the time the dump truck was being tested, a Greyhound bus was clocked through the climbing lane. On a pounds-per-horsepower basis this bus compared closely with the 11,900 pound gross load of the dump truck. The bus, however, has a higher performance curve which is due to its diesel engine. The diesel engine develops a higher horsepower at the same engine turnover per minute rate than a gasoline engine such as the dump truck had. At 850 revolutions per minute the engine of the bus develops 215 horsepower, whereas the gasoline engine of the truck must turn up 3,600 revolutions per minute in order to attain its 107 horsepower.

A long wheel base truck was found to have a performance curve similar in shape to that of a lightly laden truck with a semitrailer. The long wheel base truck pulled over and stopped at station 1379, so no comparison may be made beyond station 1393. His deceleration characteristics are somewhat faster than would be expected when compared with the 181 pounds-per-horsepower semi; however low speeds are in line at 26 and 18 miles per hour respectively. At 144 pounds per horsepower one might have expected the performance to be closely akin to that of the dump truck four ton pay load which figured 149 pounds per horsepower. The curves, however, are not similar and the minimum speed of the long wheel base is 26 miles per hour as compared with the 29 miles per hour of the dump truck. This is believed to be due to the greater tractive ability of the dump truck

because of the greater percentage of his load being directly over the rear wheels. Likewise, the 186 pounds-per-horsepower dump truck decelerates more rapidly than the 181 pounds per horsepower semi. This more rapid deceleration is due to the dump truck being in a higher gear.

curve. Acceleration for both trucks may be seen here immediately after attaining the lesser 4.56 per cent grade.

Heavily laden semitrailer trucks constitute about one-tenth of the truck traffic on this climbing lane. The two heavy trucks compared are very



However, the dump truck is six miles per hour faster at their lowest respective speeds and accelerates more rapidly on leaving the grade. As would be expected, the dump truck has a higher speed at any given point on the grade. These facts give more support to the theory of greater tractive ability for the dump truck.

The performance curves of the medium laden trucks with semitrailers (Figure 4) show a much faster deceleration for the 293 pounds-per-horsepower load than for the 213 pounds-per-horsepower load. Acceleration cannot be compared, but the explanation lies in the poor gear change of the 213 pounds-per-horsepower truck driver at about station 1382. Minimum speeds for both trucks compare well with each other and with all trucks tested of this type. Of special interest here is the conformance with the design truck curve. They are closer than were the previous trucks and at an entrance speed of 30 miles per hour, where present climbing lanes begin, both trucks very nearly coincide with the design curve. At an exit speed of 30 miles per hour, where climbing lanes end, both curves are within about a station of the design

nearly equally laden (Figure 5) at 373 and 382 pounds per horsepower respectively. No conclusions may be drawn from the acceleration or deceleration characteristics of these trucks when compared with each other. Their confliction is felt to be due to driver characteristics. However, when compared with all other trucks of this type it will be readily seen that the deceleration of these heavier loads is faster than the medium and lightly laden semis, and while the acceleration characteristics of these two trucks are quite good the low speeds are in line with the pounds carried per horsepower. Both trucks have decelerated to 30 miles per hour at station 1426, which is about 400 feet prior to the design truck speed of 30 miles per hour. A lower minimum speed than that of the design truck is encountered not only at the end of the 5 per cent grade but during some portion of all grades where the climbing lane occurs. Exit speeds were both lower than design and the 382 pounds per horsepower truck did not attain a speed of 30 miles per hour until almost 800 feet after that of the design truck.

Trucks entering this ascending

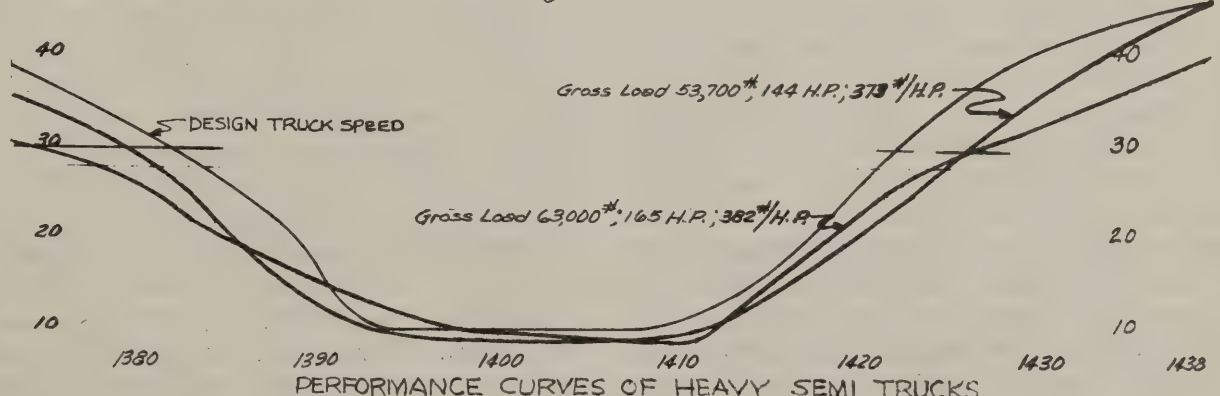
lane are somewhat slow in moving over to the right-hand edge of the pavement. This may be due to two factors. The high entrance speed contributes to this slowness, and the surface has been widened with premix giving two different colors. This two-tone surface, however, does not keep a truck from moving over when an anxious driver to the rear desires to pass soon after entrance. The author has seen the passing operation completed within two or three hundred feet of the median nose. While this is a thing that rarely happens, it is a potential danger spot and it is felt that for a future similar situation a pavement design with a minimum of 24-foot width at the median entrance should be provided. For an exit width, the 24-foot surface serves very well at station 1383. Enough sight distance is available here for passing at the median end.

Median maintenance deserves a word at this point. Openings for transverse drainage have been provided at each station within the limits of superelevated sections. For a section of the 5 per cent grade and of the 4.56 per cent grade, the surface is too lean in asphalt content. This, coupled with the fact that all traffic uses the outer lane except for passing operations, causes shed on the inner lane. These median openings within this section are partially blocked after each rain by this free aggregate. During dry seasons the gravel will stack up along side and freckle the top of the median. An asphaltic concrete pavement would have prevented this problem.

While the results of these observations are not felt by any means to be conclusive due to the limited number of tests, certain trends have been established and from these trends some conclusions may be drawn. (1) Dump trucks and busses, due to their greater tractive ability and lesser loads, will not control the beginnings or endings of climbing lanes. However the dump trucks will have low enough speeds so as to be readily passed by busses and passenger automobiles. (2) Lightly laden semis below 200 pounds per horsepower will enter the climbing lane at a slightly higher speed than the design truck, will accelerate to 30 miles per hour slightly faster than the design truck, and will have a higher minimum speed than the design truck. (3) Semis loaded in the 200 to 300 pounds per horsepower class will closely approach the design truck curve at 30 miles per hour. They too will have minimum speeds in excess of those of the design truck. (4) Heavily loaded semis above 350 pounds per horsepower will decelerate at a faster rate, and have minimum speeds slightly lower than the design truck. (5) Approach pavements should provide a minimum of 24 feet of surface at the entrance end of the median. (6) Lastly, climbing lanes speed up traffic and usually provide hazardless passing operations in hilly country.

PAVEMENT DETAILS & PROFILE OF FOUR LANE SECTION

Fig. 5



PERFORMANCE CURVES OF HEAVY SEMI TRUCKS

Problems in Horizontal Alignment of Expressways

R.A. Bossy, Expressway Engineer
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San Antonio, Texas

The subject of this talk could be rather complicated; however, after thinking it over I realize that I am asked only to discuss the problems in horizontal alignment of expressways and not to furnish the solutions. I believe that Mr. Benson, in selecting someone to give this talk, was probably influenced considerably by having seen the City of San Antonio and the expressway work there. I believe that none of the other metropolitan areas in Texas have quite as complicated a street pattern as does the City of San Antonio. Accordingly, the expressway routes in the old city are somewhat indirect.

It is rather difficult to isolate the problem of horizontal alignment in expressway work, but I believe it would be well to try to define it to some extent for purposes of this discussion. I am assuming that before the problems of horizontal alignment arise, there has been a previous route determination for the expressway in question. That is, a route through the metropolitan area from one side to the other has been selected within relatively narrow limits. With this in mind, then the horizontal alignment problem covers only the exact location along the route and the geometrics of the design, particularly as related to the interchange facilities to be provided with other highways and major traffic arteries. For determination of these features there are a number of general controls. First of these is right of way. That is, in urban work, the most important control. It is necessary to select a location on which adequate right of way can be provided at reasonable cost and one which will permit continued development of the urban area along the desired lines. Many times it is possible to utilize areas for right of way which have deteriorated for other uses or which were never well suited to other uses.

Other general controls are topography and drainage. In rural locations these are perhaps the most important features. Although these are subordinate to right-of-way problems in urban work, they are still major factors and necessarily affect the location.

The location of interchanges is of course primarily determined by intersecting highways and major crossing arteries. These points of crossing must necessarily be selected

where adequate interchange facilities can be provided and constructed at reasonable cost. We find that before a location can be made we must have a rather definite conception of the type and extent of interchange facilities which are to be provided at the various crossings. Practically all structural design in expressway work consists of individual and special cases. One should abandon any thought of forcing alignment to simplify structural design.

Another major consideration in determining expressway location should be the accessibility of the facility to the adjacent areas of the city. To serve its best purpose, the expressway should be readily accessible throughout its length. Often a location parallel and adjacent to a major stream or railroad appears to be desirable from a right-of-way standpoint. In these cases we should be careful not to overlook this feature of accessibility.

In Texas, urban expressway work is a cooperative endeavor in which the Texas Highway Department and the municipal corporations undertake the work on a cooperative basis. The city furnishes the right of way, and the state undertakes the design and construction of the expressway facility. It is unavoidable that the municipalities have many serious problems in developing an expressway route through an urban area which has developed over many years with no thought of providing such traffic facility. Most cities now have planning and zoning commissions which study their urban areas and endeavor to control the development of the cities in patterns which will lead to many benefits in the future. They have also determined what appears to be the best land use for various areas of the city, and the expressway locations must properly fit into these patterns. Also the cities in their planning have in mind the future development of major traffic arteries within the city and the expressway design must fit with this future traffic pattern. The interchange facilities other than with highways must necessarily coincide with the future traffic arteries to be developed by the city. Because of all these problems which we do not have in rural highway locations, the alignment of expressways in urban areas is an entirely different problem than rural highway alignment. It must be remembered that the primary function of an urban expressway is not to provide a facility for movement of through traffic entirely across the urban area. The smaller volume of traffic which desires to cross the urban area completely without stopping is an incidental consideration. The major function is to move traffic within the urban area, that is, traffic which enters the expressway and leaves it within the city.

Under these circumstances, and bearing in mind this major function, direct alignment for the expressway is not essential. It is rather more important to properly tap each point of traffic interchange in the best manner and to have a facility which permits the freedom of movement in the urban area, which is the objective.

To talk specifically about some of the San Antonio Expressway work, I have brought some drawings which I would like to show you regarding what has been done in that area. This map shows the urban expressway system as now proposed in San Antonio. The central section from Fredericksburg Road to Nogalitos Street and from Martin Street to Broadway is the original expressway project set up in San Antonio in 1945. Its origination was based on improvement of interstate routes in the City of San Antonio. These were U.S. 87 northwest, together with U.S. 90 east and U.S. 81 north and south. This original 6-mile project is now practically completed with the exception of the freeway lanes in the central section.

In 1952 the State Highway Commission, recognizing the need for long-range planning for expansion of the proposed expressway facilities in the metropolitan area of San Antonio, proposed to the City of San Antonio and Bexar County the ultimate development of the expressway system as indicated on the map. The City and County accepted this pro-

posal and progress has been made in its development. The designated system to be improved to expressway standards includes Loop 13 entirely around the metropolitan area, the completion of U.S. 81 across the City and also to the county line in each direction. U.S. 87 is to be extended to the Kendall County line and an East-West expressway route for U.S. 90 is to be developed. This route is shown dotted because it may be a continuous route or it may be a route for U.S. 90 west from U.S. 81 to Loop 13 and U.S. 90 east from U.S. 81 to the county line. This long-range plan in the metropolitan area alone, bounded by Loop 13, includes 49 miles of Loop 13 and 45 miles of expressway inside of Loop 13, a total of 94 miles. In looking at this over-all plan you will note that the southeast quadrant of the City, a highly developed area, is lacking in proposed expressway facilities. This fact has been under discussion for some time and studies are being made of the justification of an addition to the over-all plan in this area. This may appear to be a rather ambitious plan; however, I think the need for its completion is definite and acute. This proposal and agreement was made in 1952, two years ago. In spite of the local political wars, which have been raging continuously, Bexar County has authorized $4 million for right of way, and the City of San Antonio has authorized $11 million. This compares with $5 million spent since 1945 for right of way.

I have wandered too long from my subject. However, I thought you would be interested in the general San Antonio status. To get back to the subject I would like to give you a run-down of the general procedure which we have used in San Antonio in making expressway locations or to determine the horizontal alignment. We have first obtained aerial strip maps of, say one-third- or one-half-mile width along the proposed route. We usually get these on a scale of 1":100' and allow the map makers 2 per cent of error in scale. On these aerial maps we select the general control points in accordance with the features outlined previously. We then draw the preliminary lines to scale on these maps and outline the interchange facilities. We find that it is necessary to draw all the major features of the proposed expressway on the air map, that is the pavement lines, interchanges, ramps, and various connections. This can be done graphically without calculating alignment at this stage. Use of straightedges and curves will provide sufficient accuracy for the preliminary studies. Utilization of different colors on the expressway lanes and frontage roads and ramps assists in presenting the desired picture. After we have laid out what appears to be a reasonably satisfactory location on the aerial map we are ready to begin the field surveys. It is, of course, impossible to run center lines on this type of work and field surveys are made from base lines which are established on existing streets or other open areas. The best position for the base line is easily selected from the aerial maps. These base lines are accurately established and complete topography is obtained from the base line. We then plot this information on a coordinate grid on a scale of 1":20' on pencil tracing cloth. Prints are then made of the drawings which gives us a working map on a scale of 1":20' which shows all of the topography features. In the field surveys we also accurately tie in the property lines in the vicinity of the proposed location. I think San Antonio is no different from the other urban areas in that many of the city block maps and other available information concerning property lines have many discrepancies and exact determination of required right of way necessitates the accurate tying of property lines to the base-survey system. When these maps are completed on the 1:20 scale, control points are selected from the air map layout and the exact alignment of the proposed expressway is plotted on the map. Necessary adjustments are then made and all of the interchanges, ramps, and pavement are accurately plotted on the 1:20 layout. This large scale speeds up the work in many respects since most of the design determinations may be made

by scaling. The work may be accurately scaled within one foot. After the design is made and completed on the 1:20 layout, the required right of way is outlined on the map and these large-scale base maps are used to transmit the data to smaller scale right-of-way maps and plans.

(Studies of some of the proposed interchanges in the San Antonio Expressway system were shown here. No perspectives or charts were available for publication here, but Mr. Bossey's comments are presented as it is felt they give enough of a picture.)

This is the south intersection of U.S. 81 and Loop 13, where a cloverleaf is proposed. Remember that this is an intersection of two expressways. It will be noted that this interchange provides separate lanes for all of the interchange traffic. The through traffic is not involved with any of the right or left movements. At first glance it might appear that it would be considerably more expensive than the usual type of cloverleaf; however, I think close examination will show that the additional area of bridge and pavement required is not excessive in relation to the benefits provided.

This next interchange is U.S. 81 south and S.H. 346. This problem was not considered well suited to the cloverleaf design because the turning movements will not be of anything like equal volume. It is considered that the volume passing back and forth on S.H. 346 south will greatly exceed other movements at this interchange. Accordingly, we have developed this design which favors the anticipated heavy movements and subordinates the minor movements. Highway 346 is not a controlled-access facility.

This next interchange is U.S. 87 northwest and Loop 13, another intersection of two expressways. In this case it is considered most probably that the turning movements from northwest toward town will greatly exceed any of the other movements. We therefore provided direct connections for this movement. When this is done in one quadrant, it automatically follows in the opposite quadrant. This subordinates then the other two quadrants in this design.

Now here is a perspective drawing of the simpliest kind of interchange for two expressway crossings. It has been our thought that if U.S. 90 should be located across U.S. 81 in the central part of town right-of-way cost would prohibit any large interchange area. This would provide for complete separation of the through traffic but would require all interchange from both highways on the frontage roads at grade with signal control.

I also have here a layout of the interchange facility at the intersection of U.S. 81 and U.S. 87 in the central part of town. Most of this has been constructed and probably a number of you have seen it and I brought it along mainly for that reason to show you that some day we expect to connect things up at this intersection, even though you may have wondered about it. We will have direct connections between U.S. 81 and 87 for the north-south through traffic on both routes. The other movements will have to be made from the expressway to the frontage roads and back to the expressway.

I hope that no one in this group has expected me to come up with a formula for solution of expressway alignment problems. These are so closely related and inseparable from the many other problems involved that the subject is very difficult to talk about without involving many other features. I have come to the conclusion that the basic problems in expressway work are the same ones that we of the Highway Department have been faced with for the past 25 or 30 years, except that they are bigger now. For instance, there is the difficulty of visualizing future traffic requirements and determining the best provision to be made, as well as the difficulty of selecting from the tremendous need that part which is practical of accomplishment now and will still fit into the future pattern in a proper manner. Then there is the ever-present demand for compromising design to meet right-of-way problems, and the seemingly unavoidable necessity for right-of-way commitments in advance of design.

Selection of Pavement for Expressways

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The subject of this paper implies a decision as to the type of pavement to be used in expressway construction. As in any other item of construction, the engineer's decision must be economically sound. This means that two major items of cost must be considered and balanced against each other. These are construction cost and maintenance cost for the anticipated life of the pavement.

The first item can be estimated directly with reasonable accuracy from a knowledge of current costs of locally available materials and labor. The second item is not so easy as it involves estimating future labor and other costs and evaluation of many intangibles such as the cost to the expressway customers occasioned by interruption of service for maintenance repairs or reconstruction. The latter item can be of such magnitude as to overshadow the actual cost to the state of the work performed. For example, over 80,000 vehicles per day use the Gulf Freeway at Velasco Overpass. Should this volume of traffic be inconvenienced by maintenance operations in this vicinity in such a manner as to be delayed one minute in their travel, the cost to the users would amount to approximately $1,700 per day. A one-minute delay to all traffic could be easily occasioned by closing one of three freeway lanes in each direction for even a short distance. In addition to this cost, there are other intangible items such as danger to personnel, increased probability of accidents, and short effective work days due to necessity of avoiding work during peak hours. Work of any nature on expressway lanes of the Gulf Freeway is a most difficult and dangerous undertaking.

The public pays a good price for expressways and expects good performance. Undue interruption of service or inconvenience to the traveling public due to maintenance repairs might adversely affect public support of such facilities and we certainly cannot afford to lose the public support we now enjoy.

To date, concrete pavement has been selected for freeway lanes, service roads, and ramps for the reasons stated above, and a flexible-type pavement for parking lanes, median dividers, and some service roads. A semiflexible-type pavement has been provided in a project for a connection of the U. S. 75 North Expressway with existing streets, and a similar type may be used in some future express-

way construction. Hence, it is apparent that we are not committed to the use of concrete pavement solely in our expressway work in Houston. Each of the various types of pavement has its peculiar advantages and disadvantages and the choice in each case is governed by an analysis of construction and maintenance costs. Therefore, before proceeding into a detailed discussion of the merits of each, it may be well to review the types of pavement available and something of their methods of design.

Flexible Pavement: This type of pavement consists of one or more layers of granular material to which is applied an asphaltic wearing surface. It is distinguished from a rigid-type pavement in that the loads are carried principally by individual interaction between the particles of the granular base course. Each layer transmits a load of diminishing intensity to the underlying layer or to the subgrade, at the same time undergoing a certain amount of deflection or deformation in proportion to the loads transmitted. The resultant yielding of the pavement surface under the action of a wheel load is responsible for the term "flexible" in this type of pavement. The thickness of each layer required for stability under any given wheel load is determined from the requirement that the stress or unit load transmitted to the underlying layer or to the subgrade shall not overstress the latter. Thus it is apparent that each layer depends not only on its own strength but also on that of its next-door neighbor, and therefore the designer must know something of the characteristics of the subgrade soils and of each of the materials or mixes contemplated for use.

Desirable qualities in base and select course materials are high density, hard durable aggregate, high percentage of coarse material, correct amount of soil binder, low plasticity, and a well-balanced grading. A rough texture is also highly desirable for development of frictional resistances between particles. All of these properties contribute to the shearing resistance of the base course, which property determines its load-carrying capacity. Crushed rock, pit-run gravel, processed gravel, slag, caliche, iron ore, oyster shell, and sand-shell mixes all possess these qualities to varying degrees and are used in flexible pavements all over the state. Generally speaking, the most popular materials in any locality are those obtainable locally at less cost.

Until recently, the designer judged the qualities of each base material on the basis of its grading, past performance, plasticity index, per cent soil binder, and other physical appearances. Such tests and qualities were sufficient in a qualitative sense, but gave no hint as to the required thickness. Now, however, the Texas Highway Department has a method of testing and design that enables direct determination of the shearing resistance of subgrade soils and of any desired type of select or base course material, and a subsequent determination of the required thickness of each layer of the pavement. This method has been developed jointly by the Materials and Tests Division and Road Design Division after several years of research and has been fully described by Mr. Chester A. McDowell in papers before this Highway Short Course and also the 23rd Highway Short Course in 1949. Moreover, full details are given in Book II of the Plan Preparation Manual, Instructor's School, available to all of you, and further description in this paper would be superfluous. One comment, however, is in order in connection with a statement frequently made that on many jobs one cannot afford to perform the necessary sampling, testing, and design work. Our stand on this problem is that one cannot afford not to do this work because without such information one must either overdesign, underdesign, or make a lucky guess. Even in those situations where the design analysis indicates more thickness of base material required than can be afforded, the designer can at least make a more intelligent decision as to what element of the design to reduce or as to what other type of pavement to substitute

Popular opinion perhaps to the contrary, the urban designer does not have at his disposal an unlimited supply of funds.

Selection of a bituminous wearing course is based mainly on the availability and cost of materials. Penetration-type surfaces are not recommended for wearing surfaces for expressway work due to the limited field control possible. Some form of asphaltic concrete should be used and mixes should be designed and controlled as specified in Construction Bulletin C-11, with the added provision that a minimum Hveem Stability of 45 be specified. The flexible pavement design procedure mentioned above will give some ideas as to the thickness required, but it is felt that these design curves are too conservative as far as surfacing is concerned and choice should be made largely on the basis of past experience. It might be well to mention, however, that the Materials and Tests Division has done considerable research along the lines of applying the triaxial test procedure to asphaltic concrete mixes. As yet, the method has not proven to be suitable for field use, but it is hoped that this investigation will continue and that in the near future the design curves will be modified accordingly, so that wearing surfaces can be designed on the same basis as base and select courses.

Rigid Pavement: This type of pavement consists of a relatively nonyielding monolithic layer of portland cement concrete which acts as a single unit, rather than by interaction between individual particles. Loads are carried by beam action, the slab acting as a continuous beam on a uniform support. Wheel loads cause the slab to deflect but generally to a lesser degree than a flexible pavement. Perhaps the outstanding fundamental difference in behavior is that the concrete pavement can crack permanently with a resultant loss in strength unless the crack is properly controlled, whereas the granular nature of the flexible base course normally precludes the possibility of a permanent crack and even after cracking, the structural qualities are not impaired to the same extent.

Design of a rigid pavement involves consideration of the design wheel load, the subgrade soil on which the pavement is to be built, type and amount of subbase, if any, required to control pumping, and type of concrete to be used and the thickness required for the design wheel load.

Valuable data on the first item can be supplied by records of the Highway Planning Survey. A design wheel load of 20, 000 pounds under a tire pressure of 100 p. s. i. is now used for design of Houston's expressways. A few years ago, a 14, 000-pound wheel load and 70 p. s. i. tire pressure was used and this seemed quite conservative at the time. Study of data collected by the Planning Survey on intensity and frequency of wheel loads on Texas highways during the past several years indicates that our 20, 000-pound wheel load may soon be obsolete. In view of the fact that expressways should be designed for a minimum life of twenty years, it is quite apparent that a liberal viewpoint must be held in assigning to any project a design wheel load or traffic pattern.

The subgrade soil must be examined to determine its strength, which in the case of concrete pavement is expressed in terms of its modulus of subgrade reaction, or k-value. The latter is defined as the ratio of a unit pressure applied on a given area of the subgrade to the settlement or vertical deformation produced by the unit pressure, and has the units p. s. i. per inch. It is also evident that k-value is also numerically equal to the ratio of the total load on the given area to the volume displaced by the area during its settlement, and that the units in this expression are pounds per cubic inch. In either definition it is always defined in terms of a given area; hence, the determination of k-value must be made with a proper realization of the effect of the size of the loaded area.

The most obvious method of measuring k-value is to load steel plates of various sizes with measured loads and to measure the corresponding settlement. The full line on Figure 1 shows the results of such a series of tests

made during construction of the Gulf Freeway. The effect of size of test plates is very obvious.

Also very obvious is the disadvantage of this method of testing. It is expensive, must be done in the field, and must be performed on the prepared subgrade including a subbase if such is to be used in construction. This would require either construction of a preliminary test section or waiting until construction for the tests. The latter is comparable to getting the horse before the cart, as this information is needed before a design can be intelligently made. Fortunately, the k-value can be determined with a fair degree of accuracy by calculation from triaxial compression test data on the materials concerned, using the following simple formula:

$$k = \frac{C}{1.5\,a} \qquad (1)$$

In this formula, taken from Timoshenko's Theory of Elasticity with a slight rearrangement of terms, k refers to the k-value, a is the radius of the loaded area, and C is the elastic modulus of the subgrade determined from triaxial compression tests. Referring again to Figure 1, the broken lines show values calculated in this manner; the agreement with load test data is quite close.

One comment on this method is highly important. The term C in the above formula represents the average or composite elastic modulus of the entire soil structure on which the slab is to be placed, including natural undisturbed soil, compacted earth subgrade, and compacted subbase if such is to be provided. In addition, loads on concrete pavements affect soils to a greater depth than do similar loads on flexible pavements and in many cases the characteristics of the natural soil will change appreciably with depth. Hence, the triaxial compression tests must be made on natural soils to some depth as well as on the compacted natural soils and subbase materials, and their respective elastic moduli averaged in some manner suitable to the individual designer. One such method is by use of Figure 7, page 78 of the Plan Preparation Manual, Book II.

While this procedure may seem somewhat involved, it must be remembered that concrete pavement design is a complicated problem; also, the amount of testing required is essentially the same as required for proper design of a flexible pavement even though the interpretation of the test results is somewhat more involved. Also, as will be seen later in this paper, the k-value can vary quite widely without a large change in required thickness of slab and the process need be repeated only for drastic changes in soil conditions.

Thesubgrade soil must be examined to determine the need, if any, for a granular subbase for control of pumping. The criterion, based on grading of the soil given in page 64 of the Design Manual, appears to be the most reliable information to date and the writer has found no record of failure of this criterion to produce satisfactory results. The range of materials available for subbases is even wider than the range of materials available for base courses for flexible pavement, since even fine sands and stone screenings may suffice provided they meet the grading requirement mentioned above. In any case the material considered should be tested in its compacted state to determine its triaxial strength and its resultant effect on k-value.

In addition to preventing pumping, granular subbases provide a convenient working table for paving operations, and if properly constructed also provide a considerable degree of waterproofing of subgrade soils during construction. These qualities often result in sufficient economy to the contractor's operations to pay the cost of the subbase. Until a few years ago a sand-shell flexible base material was used as a subbase in Houston's expressway construction. This was covered with a single penetration-type seal coat to provide a wearing surface during construction. This type of base material requires considerable time for setting up, and on several occasions rains during construction penetrated the partially completed base course and compacted earth subgrade with

serious loss of compaction. In order to provide a more positive waterproofing of the subgrade and to reduce the time required for subbase construction, we have since been using a cement-stabilized sand-shell mix placed at high-water content by means of vibrated spreader-box type equipment. In intersections and other places inaccessible to this type of equipment, the material is placed at lower water content by blading and rolling. This material has much less strength than the concrete pavement slab, and may be considered to be in the category of a high-quality soil-cement. Its use eliminates the seal coat, and labor savings have resulted in a lowering of bid prices to the extent that we are now placing this type of subbase at about the same cost as the previous type with seal coat. It has been found to provide excellent protection to subgrade compaction, and no serious cracking or hair-checking has been observed. More data will be presented on this type of subbase in a later section of this paper.

Having disposed of subgrade soils and subbase, the designer must next consider the slab itself. The analysis involves the determination of tensile stresses produced in the slab by wheel loads applied to various positions on the slab, and comparison of these with the tensile or flexural strength of the concrete. Two methods are available:

1. Westergaard method based on theory of continuous beams on uniform elastic supports, and subsequent modifications.
2. Elastic theory of layered systems.

It would be highly desirable to use the layered theory method since the characteristics of the subgrade, base, and slab are all expressed in terms of Young's Modulus, which is independent of size of test specimen or size of loaded area, and which is determined directly from simple compression or triaxial compression tests. However, solutions are available only for the case of loads applied to an interior section of the slab and this load case is rarely the controlling case; also, very little data are available on stresses not directly below the center of the load. The labor involved in making numerical solutions of the appropriate equations is so enormous that anything like complete data on practical types of loads will be many years in the coming.

Hence, we must fall back on the more approximate Westergaard method and k-value. Three load cases are considered: interior, corner, and edge. For each load case, the extent of the slab affected by any given wheel load is different and the amount of subgrade affected by the deflection of the slab is also different. Recalling that k-value varies with size of loaded area, one is not surprised to learn that each load case calls for a different k-value. Here again is the familiar designer's choice, with the U. S. Engineers recommending 30-inch diameter test plates, our own Plan Preparation Manual up to 60-inch diameter, and others even larger test plates. A series of plate load tests made during construction of the Gulf Freeway on pavement subgrades and later on the slabs placed on these subgrades indicated that plate diameters of 16 and 24 inches were appropriate for edge and interior loads, respectively. Numerous design calculations have been made and these invariably show that the edge load is the most critical; hence, we have selected the sixteen-inch diameter plate size as standard equipment for k-value determination.

As is the case with flexible bases, many different materials are available for concrete construction in Texas. The elastic modulus of the material considered must be known or determined. This may vary from 1×10^6 for shell concrete to 5×10^6 for Colorado River gravel concrete. The Texas Highway Laboratory is equipped to make this determination on any concrete upon specific request, and this should be done if the elastic modulus is not known from past experience with the material in question.

Poisson's ratio for concrete is usually assumed to be 0.15; if other than gravel aggregate is used it might be well to check this property also by test.

Having assigned design values for wheel load W, radius of tire contact area a, k-value of subgrade, Young's modulus, and Poisson's ratio of concrete, various slab thicknesses, h, are selected and tensile stresses calculated for the various load cases. Before proceeding with the actual stress calculations, it is necessary to evaluate two terms that appear in all of the Westergaard formulas:

Radius of Relative Stiffness: $$l = \sqrt[4]{\frac{E\,h^3}{12\,(1-\mu^2)\,k}} \qquad (2)$$

Equivalent Radius of Resisting Sections: $$b = \sqrt{1.6a^2 + h^2} - 0.675h \qquad (3)$$

If several designs are to be investigated, it will be found convenient to plot a set of curves for each of the above quantities giving their values for various k-values, slab thicknesses, and load radii. The following listed formulas may then be used to calculate the slab stresses:

plotted from these tables. Such tables can be found in Bradbury's Reinforced Concrete Pavement, Washington, D. C., 1938. A more recent modification of the corner load case formula has been published by Pickett in Concrete Pavement Design for Roads and Streets Carrying All Classes of Traffic, Portland Cement Association, 1946. This same publication is very convenient to use as it also contains design charts giving actual stresses for the two cases of corner loads and various k-values.

The procedure just outlined gives tensile stresses due to application of a wheel load. In addition to these so-called load stresses, there may also occur tensile stresses due to

Interior Load: $$S_i = \frac{Q_i\,W}{h^2} = \frac{0.3162\,W}{h^2}\left[4\log(1/b)+0.633\right] \qquad (4)$$

Edge Load: $$S_e = \frac{Q_e\,W}{h^2} = \frac{0.572\,W}{h^2}\left[4\log(1/b)+0.359\right] \qquad (5)$$

Protected Corner: $$S_c = \frac{Q_c\,W}{h^2} = \frac{3W}{h^2}\left[1-(1.414\,a/l)^{0.6}\right] \qquad (6)$$

Unprotected Corner: $$S_c = \frac{Q_c\,W}{h^2} = \frac{3W}{h^2}\left[1-(a/l)^{0.6}\right] \qquad (7)$$

Unprotected corner case above refers to a load at a corner that is warped upward in such a manner as to reduce the k-value of the subgrade support to one-fourth its normal value. The labor involved in the calculations can be greatly simplified by using published tables of the stress coefficients Q_e, Q_c, and Q_i, or curves constrained warping of the slab. These are caused by temperature differences between the top and bottom of a slab, and also by moisture content differences. Fortunately, the two effects counterbalance each other to a certain extent and this office has used a net warping of the slab under the effect of 1.5 degrees Fahrenheit per inch

thickness of slab. These warping stresses add to the stresses induced by interior and edge loads during daylight hours and may be of considerable magnitude. In the case of the corner load, these stresses are very small due to lack of restraint at a corner and need not be considered except in its effect on reducing subgrade support at the corner, as has been done in the Unprotected Corner load formula above. In order to calculate the warping stress we first calculate the warping stress at the center of a slab of infinite horizontal directions:

$$\sigma_o = \frac{E\ e\ t}{2(1 - \mu)} \qquad (8)$$

In this formula, E is Young's modulus of the concrete, e is the coefficient of thermal expansion of the concrete, t is the estimated effective temperature difference between top and bottom of slab, and μ is Poisson's ratio. The warping stress at the center of an actual slab with finite dimensions horizontally is generally less than that of a hypothetical slab of infinite dimensions and can be evaluated by reference to Table 4 of Analysis of Stresses in Concrete Roads caused by Variations of Temperature, by H. M. Westergaard, PUBLIC ROADS, Vol. 8, No. 3, May, 1927, or to a curve plotted from the values given in the article. The values are given in terms of the ratio of σ_c, the actual stress at the center, to σ_o above. The warping stress at the edge of the slab is less than at the center due to lack of restraint in a direction perpendicular to the edge and its value can be taken as:

$$\sigma_e = \sigma_c (1 - \mu) \qquad (9)$$

As stated before, warping stresses at corner may be ignored. The warping stresses are then added to load stresses for each load case and the total stress compared with the allowable flexural strength of the concrete. The critical load case will be that load which produces the greatest total stress, and the critical pavement thickness will be the thickness that results in a stress for the critical load case that is just equal to the allowable flexural strength.

The allowable flexural strength should be based on the ultimate flexural strength expected and a safety factor of two for the design wheel load. According to the fatigue principle of concrete, such a safety factor is sufficient for an infinite number of repetitions of load. In any anticipated traffic pattern there will be a very small percentage of the total traffic that will exceed the design wheel load, and for these loads a lower safety factor may be used, depending on the anticipated number of repetitions. Figure 7, shown on page 34 of Concrete Pavement Design for Roads and Streets Carrying All Classes of Traffic, Portland Cement Association, 1946, furnishes data on safety factors required for any anticipated number of repetitions.

The author realizes that many designers do not consider warping stresses in concrete pavement design. This is a valid procedure for the corner load case in which the warping stress is small. However, our experience with these design calculations is that the edge stress is always greater than either corner or interior stress; hence, we feel we must try to evaluate them. In addition, we have some very strong evidence that warping stress has been responsible for most of the transverse cracking observed to date on the Gulf Freeway; the total amount of such cracks is small, but the amounts observed in sections having 30-foot joint spacing is much greater than in similar sections with 20-foot spacing. The unfortunate aspect of this problem is that our information on warping stresses in actual slabs is very meager and is based mainly on observations of slabs eight inches and less in thickness. Our present procedure is to assume a direct proportion between thickness and temperature differential. Hence, when we make calculations for wheel loads of 20,000 pounds and more we often arrive at unreasonable answers for thicker slabs. It is the author's opinion that the temperature differential becomes constant for all slabs over a certain

thickness, but there is no information available on this matter. It is strongly urged that someone initiate research on this problem.

The subject of other physical aspects of design such as reinforcing, if any, joints, and joint spacing is the subject of other papers before this Short Course and will not be repeated here. However, it is evident from the above discussion that joint spacing can materially affect the thickness design and its effect on thickness should be investigated.

Continuously Reinforced Concrete Pavement: This is another type of rigid pavement which merits discussion here. In this type of pavement joints are eliminated by supplying sufficient reinforcing steel to bond the steel and concrete together and to develop a pattern of closely spaced cracks with very minute openings. The cracks are produced by a combination of bond between steel and concrete and elongation of the steel under stress to a greater extent than the concrete. Some of this type of pavement has been used on Fort Worth expressways with considerable success. I shall be interested to hear some discussion on this point by Mr. Lovett in a few minutes. We have investigated this type of design and have found some reduction in thickness possible in our situation; the cost appeared to be about the same as ordinary concrete pavement with joints. We have not tried any of this in our construction as our present feeling is that we can achieve the same result — closely spaced joints with small openings — with ordinary pavement and with more control over the joints or cracks by using about a fifteen-foot joint spacing.

Shell Concrete: Shell concrete is a portland cement concrete constructed by using oyster shell for coarse aggregate. It has been used in many State Projects in the coastal region where the shell aggregate is the only local aggregate available in large quantities and at low cost. District 12 has pioneered in development and use of this type of pavement. This development is a good example of the relentless efforts of engineers of the Texas Highway Department in attempting to find uses for local materials and hence cut construction costs.

Shell concrete has generally less strength than gravel concrete, but at the same time possesses some other qualities that are superior to gravel concrete, namely 1) Low elastic modulus, 2) Low coefficient of thermal expansion, and 3) Greater strength horizontally than vertically.

The low elastic modulus results in lower load stresses, thus partially counterbalancing its lower strength. This may seem to be a startling statement but may be explained on the basis of continuous beam theory on uniform supports and can be confirmed by application of the Westergaard stress formulas given in the previous section. The practical explanation of this is that the stiffer or more rigid the concrete is, compared to the subgrade on which it is placed, the more load it will carry and hence the higher will be the stress. Conversely, the more flexible the concrete, the more load will be transmitted to and carried by the subgrade.

The low temperature coefficient reduces the amount of warping due to temperature differentials. The low elastic modulus further reduces the warping stress due to any amount of warping. Hence, warping stresses are reduced to a very low value. This fact may be confirmed immediately by examination of Formula 8 above, wherein both elastic modulus and temperature coefficient appear in the upper half of the fraction. With elastic modulus about one-fifth that of gravel concrete and temperature coefficient about one-half, it is apparent that warping stress for any given temperature differential and slab thickness is about one-tenth as great in shell concrete as in gravel concrete. Another most important circumstance of the low temperature coefficient is that opening and closing of joints is markedly less; hence, longer joint spacing can be used, or with the same joint spacing joint openings will be much less. This fact can be easily seen in any shell concrete pavement. The smaller joint openings are very very evident.

The third peculiar property listed above may come as a surprise to some. Due to the predominantly flat or horizontal orientation of the shell aggregate in a shell concrete, the elastic modulus is higher and the temperature coefficient lower in the horizontal direction than vertically. Moreover, the overlapping of flat particles in the mix allows greater deflection of the slab without damage than is possible with a gravel concrete. This quality is considered to be an advantage when placed over the relatively soft to medium clays of the coastal region.

Unfortunately, shell concrete does not ordinarily have a hard-wearing surface and must be provided with a bituminous wearing course. It is possible that further improvement of mixes and the use of some form of concrete additive might result in a satisfactory wearing surface, but to date such has not been accomplished. Another slight disadvantage is difficulty of field control, due principally to variations in shell grading. The high voids content of shell aggregate requires generally more mixing water (average about 10 to 11 gal./sack) which fact accounts for its lower strength. Another important difference is its lower rate of strength increase, and this fact sounds a warning against basing design of seven-day beam strengths. Instances have been cited of a 100 per cent increase in strength of shell concrete after 28 days.

In spite of the disadvantages mentioned above, shell concrete offers some very desirable qualities and these should be investigated in any locality in which this type of aggregate is available at low cost.

Other Low-modulus Concrete Pavements: There are other types of concrete that have properties similar to shell concrete, and one or more of these types may be worthy of consideration in other than coastal regions. Many cities use on their streets a type of pavement in which a rather weak gravel concrete with little or no reinforcing steel or joints is overlaid with a thin bituminous wearing surface. The higher than normal water-cement ratio apparently reduces the E and e somewhat and places this type of pavement in the category under discussion. Another possibility which to the author's knowledge has not been used or considered for pavement is lightweight concrete. Most of the lightweight concretes have a low elastic modulus and some are claimed to provide a good wearing surface. They have a low thermal conductivity and should accordingly have a low coefficient of thermal expansion. It is reported that floor slabs constructed of lightweight concrete do not show shrinkage cracks to the extent that gravel concrete does. Flexural strengths of 600 p.s.i. at 28 days have been reported. It appears to the author that this is a field of possibility that should be investigated.

Semi-flexible Type Pavement: There is available a third type of pavement which is neither truly flexible nor truly a rigid type. Included in this category are pavements constructed with bases of soil-cement, soil-lime, cement-stabilized sand shell, and lime-stabilized gravel

TEXAS HIGHWAY DEPARTMENT — HOUSTON URBAN EXPRESSWAYS

Office of the Engineer-Manager, Houston, Texas

FORMULAS FOR RIGID PAVEMENT DESIGN

K-value: $$k = \frac{C}{1.5\,a} \qquad (1)$$

Radius of Relative Stiffness: $$l = \sqrt[4]{\frac{E\,h^3}{12(1-\mu^2)\,k}} \qquad (2)$$

Equivalent Radius of Resisting Section: $$b = \sqrt{1.6\,a^2 + h^2} - 0.675\,h \qquad (3)$$

STRESSES FOR LOAD AT:

Interior: $$S_i = \frac{Q_i W}{h^2} = \frac{0.3162\,W}{h^2}\left[4\log l/b + 0.633\right] \qquad (4)$$

Edge: $$S_e = \frac{Q_e W}{h^2} = \frac{0.572\,W}{h^2}\left[4\log l/b + 0.359\right] \qquad (5)$$

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Unprotected Corner: $$S_c = \frac{Q_c W}{h^2} = \frac{3\,W}{h^2}\left[1 - (a/l)^{0.6}\right] \qquad (7)$$

WARPING STRESSES:

Infinite Slab: $$\sigma_o = \frac{E\,e\,t}{2(1-\mu)} \qquad (8)$$

Center of Finite Slab: σ_c - See "Public Roads", Vol.8, No.3, May 1927

Edge of Finite Slab: $$\sigma_e = \sigma_c\,(1-\mu) \qquad (9)$$

TERMS AND UNITS:

C - Elastic Modulus of Pavement Subgrade, psi/in/in
a - Radius of loaded area of subgrade in Formula (1), inches
Radius of tire contact area in Formulas (3) thru (7), inches
E - Elastic Modulus of Concrete, psi/in/in
h - Thickness of Slab, inches
μ - Poisson's Ratio of Concrete
W - Design Wheel Load, lbs
e - Coefficient of Thermal Expansion of Concrete, in/in/°F.
t - Temperature Differential in Slab, °F

bases. These materials derive a considerable amount of their strength from some form of cementing agent and are monolithic to the extent of the strength of the cementing agent. Their load carrying action cannot be considered to be that of a granular material and the strength of any of these materials will greatly exceed that required for classification as Class I base material on the Classification Chart used for designing flexible bases. At the same time such materials cannot be classed as rigid pavement due to their method of construction, low elastic modulus, and other physical aspects. All require an asphaltic wearing surface, and for this reason have been treated and designed as flexible-type pavements. There are those among us who think that such should be considered as a weak concrete slab and designed in the same manner as a rigid pavement. Perhaps the best procedure is to analyze such a pavement according to both procedures and to rely on the method of design and indicates the least safety factor.

Since we are considering here a hybrid type of pavement having some of the characteristics of flexible and some of the characteristics of rigid pavement, it would be desirable and interesting to have a correlation of the engineering characteristics used in both methods of design. Such a correlation has been compiled during the past several years through a long-term study of concrete pavement placed on the Gulf Freeway, shell concrete placed by District 12, and cement-stabilized sand-shell subbase for concrete pavement on various Houston Expressways. Table 1 summarizes the data collected and the curves of Figure 2 show some of the more essential engineering characteristics graphically. As can be seen by the curves, there is a reasonable transition of properties from truly rigid pavement (gravel concrete) in the left side of the figures, through shell concrete and cement-stabilized sand-shell subbase, and on to sand shell on other base materials on the extreme right.

Many interesting facts may be surmised from the curves of Figure 2. For example, some of the promising characteristics of shell concrete previously discussed are clearly revealed in that while its elastic modulus is only about 25 per cent that of gravel concrete, its flexural strength is about 75 per cent that of gravel concrete. Also, the manner in which the ratio between compressive and flexural strength varies with the water-cement ratio, the value being around six to one for gravel concrete and around two to one for cement-stabilized sand-shell base. Projecting the curves on the lower half of the figure to an ordinate representing an infinite water-cement ratio (zero cement content) it appears that a first-class flexible base material should have a flexural strength almost equal to its compressive strength. While it has been commonly assumed that soils and base materials cannot carry tensile stresses, it is known that all compacted flexible base materials possess some cohesive strength when tested in triaxial compression. Moreover, theoretical considerations often indicate the presence of tensile stresses in flexible base materials without failure. While this part of the discussion may be somewhat off the subject, I have taken the liberty of mentioning it in order to show the orderly progression of properties from gravel concrete to granular base materials.

Figure 3 shows coefficients of thermal expansion of some of these materials as well as the rate of strength increase, expressed as the ratio between 28- and 7-day strength. While the data on these two characteristics is not as complete as those shown on Figure 2, the basic trends are evident. Both of these characteristics are lower for the wetter mixes. This means that our search for a pavement without joints must be confined to the higher water-cement ratio range, and that in all probability design of such a pavement will not be based on 7-day strengths and possibly not even on 28-day strengths.

From the standpoint of cracking, jointing, and temperature effects, our semiflexible pavement should have a

TEXAS HIGHWAY DEPARTMENT — HOUSTON URBAN EXPRESSWAYS

Office of the Engineer-Manager, Houston, Texas

TABLE I

CHARACTERISTICS OF PAVEMENT MATERIALS

Summary of average test results and other data on gravel concrete, shell concrete, cement-stabilized sand shell, and sand-shell flexible base.

MATERIAL	GRAVEL CONCRETE	SHELL CONCRETE	CEMENT-STABILIZED SAND-SHELL			SAND-SHELL FLEX. BASE
CONTROL NUMBER	500-3	508-1-4	500-3-26	271-14-5	177-11-2 177-11-3	500-3
HIGHWAY	US 75	State 73	US 75	Loop 137	US 59	US 75
CEMENT, sks/cy	5.0	5.0	2.8	2.8	2.8	0
TOTAL WATER, gal/sk	6.0	10.5	13.7	19.6	21.7	∞
METHOD OF PLACEMENT	Formed	Formed	Rolled	Vibrated Wet	Vibrated Wet	Rolled
TOTAL(WET)DENSITY, lb/cf	150	141	135	126	122	137
DRY DENSITY, lb/cf		130	125	115	111	122
MOISTURE CONTENT, %		8.5	7.6	10.3	11.6	12.3
AIR VOIDS, %		9	11	16	17	6
ABSORPTION, %		9.5	9.0	11.0	12.3	
ULTIMATE COMPRESSIVE STRENGTH, psi	5000	1800 *	1290	618	433	120
ULTIMATE ELASTIC MODULUS, psi/in/in	4,000,000	800,000	168,000	73,000	45,000	20,000
ULTIMATE FLEXURAL STRENGTH, psi	900	600	423	285	207	
RATIO OF COMPRESSIVE TO FLEXURAL STRENGTH	6.2	3.0	3.05	2.26	2.10	
RATIO OF 28-day TO 7-day STRENGTH	1.38			1.69	1.89	
COEFFICIENT OF THERMAL EXPANSION, in/in/°F,x10⁶	8.0	6.5**			3.5	

* 1,400,000 Horizontally ** 4.0 Horizontally

August 1954

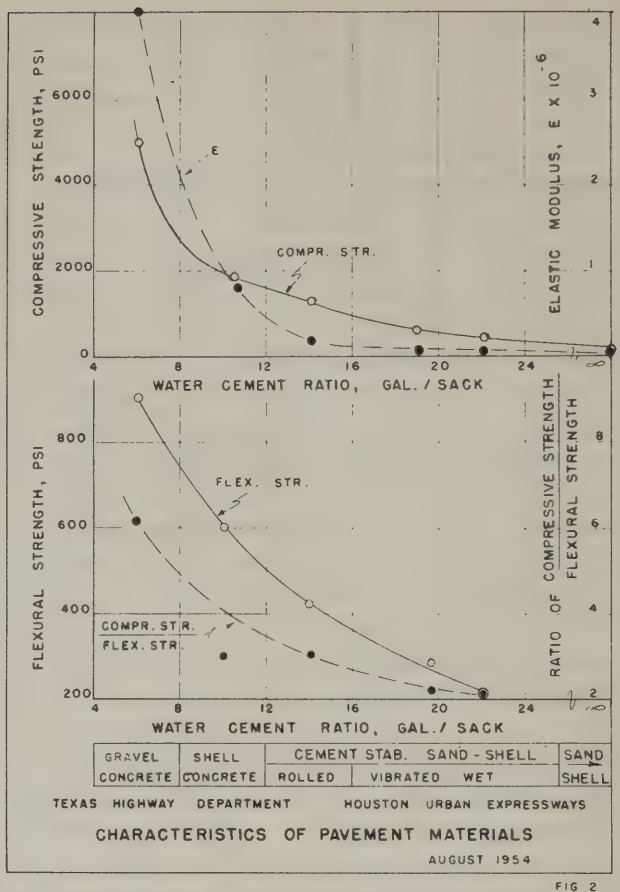

FIG 2

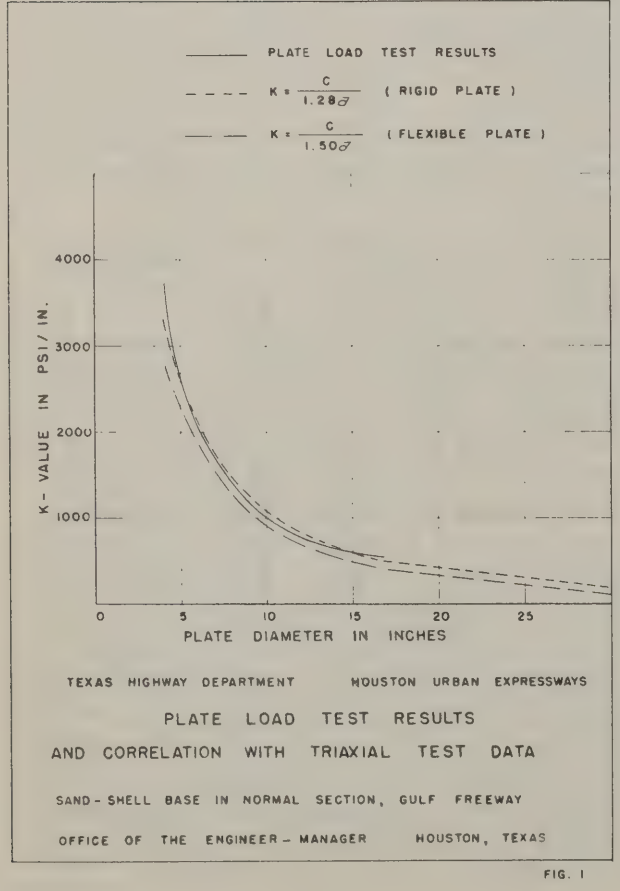
PLATE LOAD TEST RESULTS
K = C / 1.28σ (RIGID PLATE)
K = C / 1.50σ (FLEXIBLE PLATE)
K - VALUE IN PSI / IN.
PLATE DIAMETER IN INCHES
TEXAS HIGHWAY DEPARTMENT HOUSTON URBAN EXPRESSWAYS
PLATE LOAD TEST RESULTS
AND CORRELATION WITH TRIAXIAL TEST DATA
SAND - SHELL BASE IN NORMAL SECTION, GULF FREEWAY
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FIG. 1

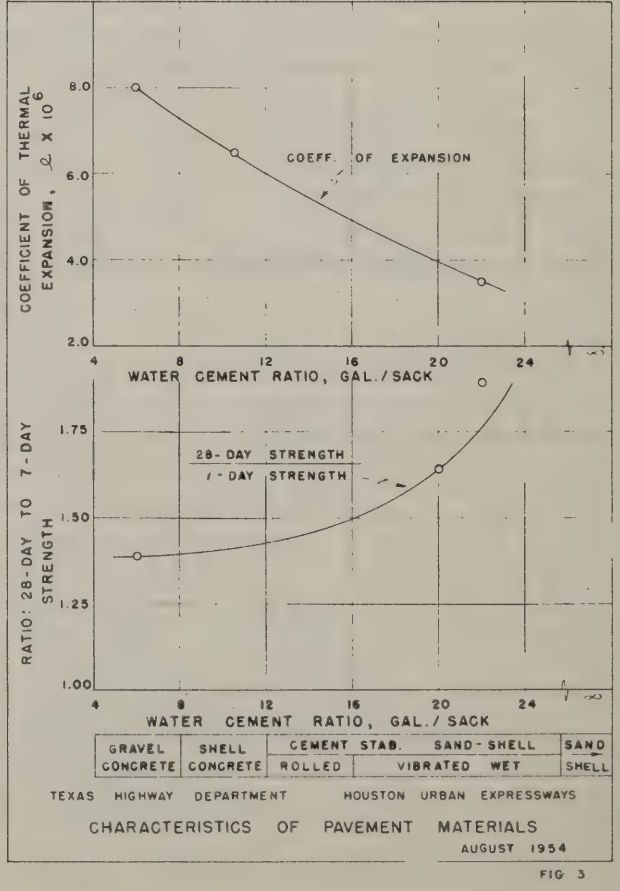

FIG 3

low elastic modulus and temperature coefficient. From the standpoint of load-carrying capacity it should have as great strength as possible. Hence the problem appears to be the selection of a water-cement ratio as high as possible without danger of permanent crack formation.

The determination of the boundary between the category of a rigid pavement material and that of semiflexible base materials requiring no joints or load-transfer devices has been the subject of considerable study by the California Department of Highways. They report in an article by Earl Withycombe (see Highway Research Abstracts, Vol. 24, No. 2, February, 1954) that soil-cement bases with 7-day compressive strengths of up to 650 p. s. i. showed no objectionable cracking. Our experience with cement-stabilized sand-shell base materials indicates that this should correspond to 28-day compressive strengths of 1,105 p. s. i. and ultimate strength of approximately 1,200 p. s. i. On this basis it appears that all of our cement-stabilized sand-shell base could fall into the semiflexible category, even that placed at relatively low water-cement ratio by blading and rolling. We are attempting to further study this problem by investigating the coefficient of thermal expansion of this material and comparing these with the coefficients for definitely rigid-type materials like shell concrete and gravel concrete. In the meantime it is believed that the criterion of 1, 200 p. s. i. ultimate compressive strength can be used as an approximate limit for materials not requiring crack control. For cement-stabilized sand-shell, this strength will be attained with approximately fourteen gallons water per sack, with a resulting coefficient of expansion of approximately 5×10^{-6}.

As pointed out previously, all semiflexible pavement materials will classify as Class I base material using the Flexible Pavement Design Chart, and hence the latter is of little value in thickness determinations. By use of rigid-design procedure and correlations of physical properties such as those depicted in Figures 2 and 3, one may differentiate between requirements for materials in the semiflexible range on the basis of their compressive strength. This covers a range of approximately 150 to 1, 200 p. s. i. It is obvious that such differentiation is essential.

Subgrades for Semiflexible Pavements: Rigid design procedure, however, does not furnish any information on the ability of the subgrade to carry the loads transmitted to it. In the case of truly rigid pavements this is not a factor to be concerned with as the high strength of the pavement is sufficient to result in very low intensity of stress transmitted to the subgrade. As pointed out in the discussion of shell concrete, the weaker the concrete, the greater the amount of load transmitted to the subgrade. Therefore, in considering semiflexible pavements, we must investigate the intensity of stresses transmitted to the subgrade. For this purpose, flexible design procedure supplies the necessary information.

Summary of Design Procedure for Semiflexible Pavements: Since the range of strengths of semiflexible pavement materials covers a wide range, it is recommended that thickness requirements be determined by application of rigid pavement design procedure. In addition, the ability of the subgrade soils to carry the loads transmitted thereto should be checked by application of flexible design procedure to the subgrade. The question of a satisfactory wearing surface for semiflexible pavements is still an unsolved problem. Three possible methods of solution appear to be:

1. Improvement in either techniques of design or in composition of bituminous wearing surfaces (by additives of rubber, etc.) to the extent that shoving, raveling, and peeling will be eliminated.

2. Improvement of wearing qualities of semiflexible pavement materials by use of additives or improvement of design mixes to the extent that excessive abrasion will be eliminated.

3. Hard aggregate mat surfacing worked into surface of freshly placed material.